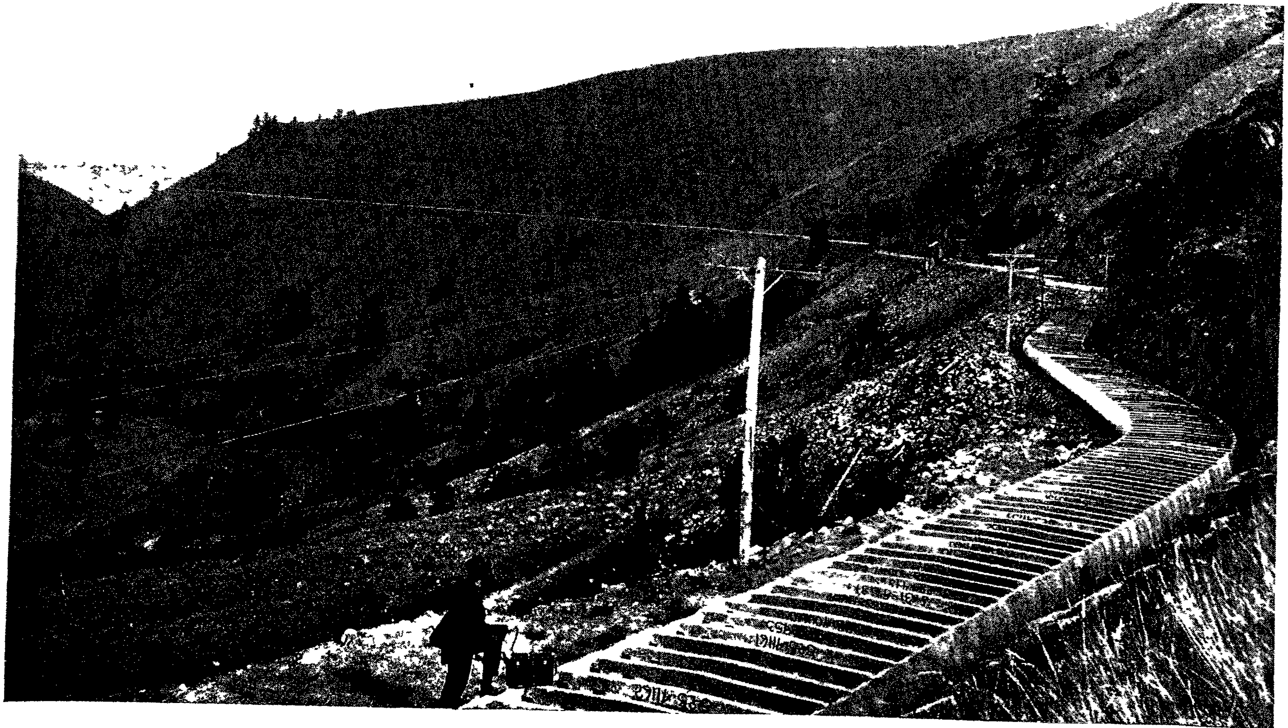


CONCRETE CANAL, PART OF THE YAKIMA VALLEY IRRIGATION PROJECT

Photo by Brown Brothers, New York City

Cyclopedia

of

Civil Engineering

A General Reference Work on

SURVEYING, HIGHWAY CONSTRUCTION, RAILROAD ENGINEERING, EARTHWORK, STEEL CONSTRUCTION, SPECIFICATIONS, CONTRACTS, BRIDGE ENGINEERING, MASONRY AND REINFORCED CONCRETE, MUNICIPAL ENGINEERING, HYDRAULIC ENGINEERING, RIVER AND HARBOR IMPROVEMENT, IRRIGATION ENGINEERING, COST ANALYSIS, ETC.

CIVIL AND CONSULTING ENGINEERS AND TECHNICAL EXPERTS OF THE ~~HIGHEST~~ PROFESSIONAL ~~STANDING~~

Illustrated with over Two Thousand Engravings

NINE VOLUMES

AMERICAN TECHNICAL SOCIETY
CHICAGO
1921

Authors and Collaborators

FREDERICK E TURNEAURE, C E., Dr Eng.

Dean of the College of Engineering, and Professor of Engineering, University of Wisconsin
Member, American Society of Civil Engineers
Joint Author of "Principles of Reinforced Concrete Construction," "Public Water Supplies," etc.

FRANK O DUFOUR, C. E.

With Stone and Webster, Boston, Massachusetts
Formerly Structural Engineer with Interstate Commerce Commission
Formerly Assistant Professor of Structural Engineering, University of Illinois
Member, American Society of Civil Engineers
Member, American Society for Testing Materials

WALTER LORING WEBB, C E.

Consulting Civil Engineer
Member, American Society of Civil Engineers
Author of "Railroad Construction," "Economics of Railroad Construction," etc

W G BLIGH

Inspecting Engineer of Irrigation Works, Department of Interior, Canada
Formerly in Engineering Service of His Majesty in India
Member, Institute Civil Engineers (London)
Member, American Society of Civil Engineers
Member, Canadian Society of Civil Engineers

ADOLPH BLACK, C. E.

Civil and Sanitary Engineer, General Chemical Company, New York City
Formerly Adjunct Professor of Civil Engineering, Columbia University

EDWARD R. MAURER, B C. E.

Professor of Mechanics, University of Wisconsin
Joint Author of "Principles of Reinforced Concrete Construction"

AUSTIN T. BYRNE

Civil Engineer
Author of "Highway Construction," "Materials and Workmanship"

A MARSTON, C. E.

Dean of Division of Engineering and Professor of Civil Engineering, Iowa State College
Member, American Society of Civil Engineers
Member, Western Society of Civil Engineers

De WITT V. MOORE

Consulting Engineer and Architect
Formerly District Engineer—Central District Division of Valuation
Interstate Commerce Commission, Chicago
Member, American Society of Engineering Contractors
Member, Indiana Engineering Society

W HERBERT GIBSON, B. S, C. E

Civil Engineer
Designer of Reinforced Concrete

JAMES K. FINCH, C. E

Associate Professor of Civil Engineering, and Director of Summer School of Surveying, Columbia University, New York

HENRY J BURT, B. S, C E

General Manager for Holabird and Roche, Architects
Member, American Society of Civil Engineers
Member, Western Society of Civil Engineers
Member, Society for the Promotion of Engineering Education

RICHARD I. D ASHBRIDGE

Civil Engineer
Member, American Society of Civil Engineers

HERMAN K. HIGGINS

Civil Engineer
Associate Member, American Society of Civil Engineers
Member, Boston Society of Civil Engineers
Member, New England Water Works Association
Member, American Railway Bridge and Building Association

ALFRED E. PHILLIPS, C. E., Ph. D.

Professor of Civil Engineering, Armour Institute of Technology

H. E. MURDOCK, M. E, C. E

Head of Department of Agricultural Engineering, Montana State College, Bozeman, Montana
Formerly Irrigation Engineer, U S Department of Agriculture

A. B. McDANIEL, B. S.

Formerly Assistant Professor of Civil Engineering, University of Illinois
Member, American Society of Civil Engineers
Member, Society for the Promotion of Engineering Education
Fellow, Association for the Advancement of Science
Author of "Excavating Machinery"

GLENN M HOBBS, Ph D

Secretary and Educational Director, American School of Correspondence
Formerly Instructor, Department of Physics, University of Chicago
American Physical Society

THOMAS FLEMING, Jr, B S, C. E.

With Chester & Fleming, Hydraulic and Sanitary Engineers
Associate Member, American Society of Civil Engineers
Member, New England Water Works Association
Member, Engineers' Society of Pennsylvania

CHARLES E MORRISON, C E, Ph D

Formerly Instructor in Civil Engineering, Columbia University
Associate Member, American Society of Civil Engineers
Author of "Highway Engineering," "High Masonry Dam Design"

EDWARD B WAITE

Formerly Dean, and Head, Consulting Department, American School of Correspondence
American Society of Mechanical Engineers
Boston Society of Civil Engineers

C. A. MILLER, Jr.

Associate Editor, American Technical Society
Affiliated Member, Western Society of Engineers
Member, American Association of Engineers
Member, Illinois Society of Architects

JESSIE M. SHEPHERD, A B.

Head, Publication Department, American Technical Society

Authorities Consulted

THE editors have freely consulted the standard technical literature of America and Europe in the preparation of these volumes. They desire to express their indebtedness, particularly, to the following eminent authorities, whose well-known treatises should be in the library of everyone interested in Civil Engineering.

Grateful acknowledgment is here made also for the invaluable co-operation of the foremost Civil, Structural, Railroad, Hydraulic, and Sanitary Engineers and Manufacturers in making these volumes thoroughly representative of the very best and latest practice in every branch of the broad field of Civil Engineering.

WILLIAM G. RAYMOND, C. E.

Dean of the School of Applied Science and Professor of Civil Engineering in the State University of Iowa; American Society of Civil Engineers
Author of "A Textbook of Plane Surveying," "The Elements of Railroad Engineering"

JOSEPH P. FRIZELL

Hydraulic Engineer and Water-Power Expert, American Society of Civil Engineers
Author of "Water Power, the Development and Application of the Energy of Flowing Water"

FREDERICK E. TURNEAURE, C. E., Dr. Eng.

Dean of the College of Engineering and Professor of Engineering, University of Wisconsin
Joint Author of "Public Water Supplies," "Theory and Practice of Modern Framed Structures," "Principles of Reinforced Concrete Construction"

HENRY N. OGDEN, C. E.

Professor of Sanitary Engineering, Cornell University
Author of "Sewer Design"

DANIEL CARHART, C. E.

Emeritus Professor of Civil Engineering, University of Pittsburgh
Author of "Treatise on Plane Surveying"

HALBERT P. GILLETTE

Editor of *Engineering and Contracting*, American Society of Civil Engineers, Formerly Chief Engineer, Washington State Railroad Commission
Author of "Handbook of Cost Data for Contractors and Engineers"

CHARLES E. GREENE, A. M., C. E.

Late Professor of Civil Engineering, University of Michigan
Author of "Trusses and Arches, Graphic Method," "Structural Mechanics"

A. PRESCOTT FOLWELL

Editor of *Municipal Journal and Engineer*, Formerly Professor of Municipal Engineering, Lafayette College
Author of "Water Supply Engineering," "Sewerage"

IRVING P. CHURCH, C. E.

Professor of Applied Mechanics and Hydraulics, Cornell University
Author of "Mechanics of Engineering"

PAUL C. NUGENT, A. M., C. E.

Professor of Civil Engineering, Syracuse University
Author of "Plane Surveying"

FRANK W. SKINNER, C. E.

Consulting Engineer; Associate Editor of *The Engineering Record*
Author of "Types and Details of Bridge Construction"

HANBURY BROWN, K. C. M. G.

Member of the Institution of Civil Engineers
Author of "Irrigation, Its Principles and Practice"

SANFORD E. THOMPSON, S. B., C. E.

American Society of Civil Engineers
Joint Author of "A Treatise on Concrete, Plain and Reinforced"

JOSEPH KENDALL FREITAG, B. S., C. E.

American Society of Civil Engineers
Author of "Architectural Engineering," "Fireproofing of Steel Buildings," "Fire Prevention and Fire Protection"

AUSTIN T. BYRNE, C. E.

Civil Engineer
Author of "Highway Construction," "Inspection of Materials and Workmanship Employed in Construction"

JOHN F. HAYFORD, C. E.

Expert Computer and Geodesist, U. S Coast and Geodetic Survey
Author of "A Textbook of Geodetic Astronomy"

WALTER LORING WEBB, C. E.

Consulting Civil Engineer; American Society of Civil Engineers
Author of "Railroad Construction in Theory and Practice," "Economics of Railroad Construction," etc.

EDWARD R. MAURER, B. C. E.

Professor of Mechanics, University of Wisconsin
Joint Author of "Principles of Reinforced Concrete Construction"

HERBERT M. WILSON, C. E.

Geographer and Former Irrigation Engineer, United States Geological Survey, American Society of Civil Engineers
Author of "Topographic Surveying," "Irrigation Engineering," etc.

MANSFIELD MERRIMAN, C. E., Ph. D.

Consulting Engineer
Formerly Professer of Civil Engineering, Lehigh University
Author of "The Elements of Precise Surveying and Geodesy," "A Treatise on Hydraulics," "Mechanics of Materials," "Retaining Walls and Masonry Dams," "Introduction to Geodetic Surveying," "A Textbook on Roofs and Bridges," "A Handbook for Surveyors," "American Civil Engineers' Pocket Book"

DAVID M. STAUFFER

American Society of Civil Engineers; Institution of Civil Engineers, Vice-President, Engineering News Publishing Co.
Author of "Modern Tunnel Practice"

CHARLES L. CRANDALL

Professor of Railroad Engineering and Geodesy in Cornell University
Author of "A Textbook on Geodesy and Least Squares"

N. CLIFFORD RICKER, M. Arch.

Professor of Architecture, University of Illinois; Fellow of the American Institute of Architects and of the Western Association of Architects
Author of "Elementary Graphic Statics and the Construction of Trussed Roofs"

W. H. SEARLES, C. E.

Author of "Field Engineering" and "Railroad Spiral"

HENRY T. BOVEY

Late Rector of Imperial College of Science and Technology, London, England
Author of "Treatise on Hydraulics"

WILLIAM H. BIRKMIRE, C. E.

Author of "Planning and Construction of High Office Buildings," "Architectural Iron and Steel, and Its Application in the Construction of Buildings," "Compound Riveted Girders," "Skeleton Structures," etc.

IRA O. BAKER, C. E.

Professor of Civil Engineering, University of Illinois

Author of "A Treatise on Masonry Construction," "Engineers' Surveying Instruments, Their Construction, Adjustment, and Use," "Roads and Pavements"

JOHN CLAYTON TRACY, C. E.

Assistant Professor of Structural Engineering, Sheffield Scientific School, Yale University

Author of "Plane Surveying: A Textbook and Pocket Manual"

FREDERICK W. TAYLOR, M. E.

Joint Author of "A Treatise on Concrete, Plain and Reinforced"

J. B. JOHNSON, C. E.

Author of "Materials of Construction," Joint Author of "Design of Modern Frame Structures"

FRANK E. KIDDER, C. E., Ph. D.

Consulting Architect and Structural Engineer; Fellow of the American Institute of Architects

Author of "Architect's and Builder's Pocketbook," "Building Construction and Superintendence, Part I, Masons' Work; Part II, Carpenters' Work, Part III, Trussed Roofs and Roof Trusses," "Strength of Beams, Floors, and Roofs"

WILLIAM H. BURR, C. E.

Professor of Civil Engineering, Columbia University; Consulting Engineer; American Society of Civil Engineers, Institution of Civil Engineers

Author of "Elasticity and Resistance of the Materials of Engineering," Joint Author of "The Design and Construction of Metallic Bridges," "Suspension Bridges, Arch Ribs, and Cantilevers"

WILLIAM M. GILLESPIE, LL. D.

Formerly Professor of Civil Engineering in Union University

Author of "Land Surveying and Direct Leveling," "Higher Surveying"

GEORGE W. TILLSON, C. E

Past President of the Brooklyn Engineers' Club, American Society of Civil Engineers; American Society of Municipal Improvements

Author of "Street Pavements and Street Paving Material"

CHARLES E. FOWLER

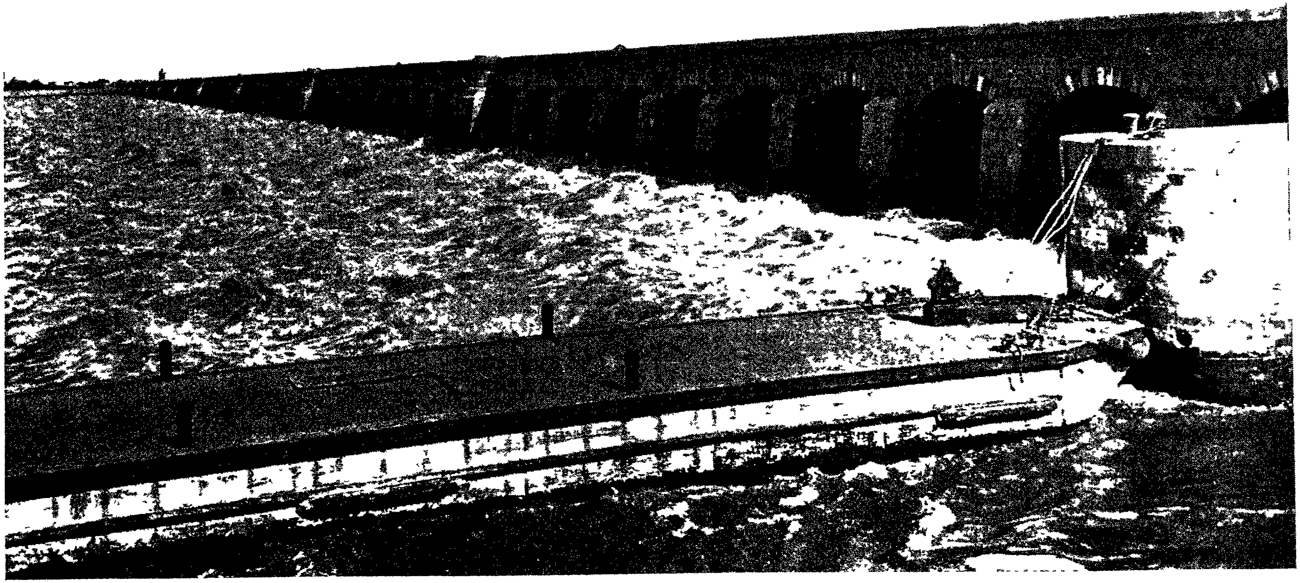
Consulting Civil Engineer, Member, American Society of Civil Engineers

Author of "Practical Treatise on Subaqueous Foundations"

W. M. PATTON

Late Professor of Engineering at the Virginia Military Institute

Author of "A Treatise on Civil Engineering"



VIEW OF RECONSTRUCTED ASSOUAN DAM ACROSS THE NILE RIVER

This dam, located just above the first cataract, is 6,400 feet long and was recently raised about 23 feet, thus increasing the amount of water impounded to over 1½ million acre feet.

Photo by Underwood and Underwood, New York City

Foreword

OF all the works of man in the various branches of engineering, none are so wonderful, so majestic, so awe-inspiring as the works of the Civil Engineer. It is the Civil Engineer who throws a great bridge across the yawning chasm which seemingly forms an impassable obstacle to further progress. He designs and builds the skeletons of steel to dizzy heights, for the architect to cover and adorn. He burrows through a great mountain and reaches the other side within a fraction of an inch of the spot located by the original survey. He scales mountain peaks, or traverses dry river beds, surveying and plotting hitherto unknown, or at least unsurveyed, regions. He builds our Panama Canals, our Arrow Rock and Roosevelt Dams, our water-works, filtration plants, and practically all of our great public works.

¶ The importance of all of these immense engineering projects and the need for a clear, non-technical presentation of the theoretical and practical developments of the broad field of Civil Engineering has led the publishers to compile this great reference work. It has been their aim to fulfill the demands of the trained engineer for authoritative material which will solve the problems in his own and allied lines in Civil Engineering, as well as to satisfy the desires of the self-taught practical man who attempts to keep up with modern engineering developments.

¶ Books on the several divisions of Civil Engineering are many and valuable, but their information is too voluminous to be of the greatest value for ready reference. The Cyclopedia of Civil Engineering offers more condensed and less technical treatments of these same subjects from which all unnecessary duplication has been eliminated; when compiled into nine handy volumes, with comprehensive indexes to facilitate the looking up of various topics, they represent a library admirably adapted to the requirements of either the technical or the practical reader.

¶ The Cyclopedia of Civil Engineering has for years occupied an enviable place in the field of technical literature as a standard reference work and the publishers have spared no expense to make this latest edition even more comprehensive and instructive.

¶ In conclusion, grateful acknowledgment is due to the staff of authors and collaborators—engineers of wide practical experience, and teachers of well recognized ability — without whose hearty co-operation this work would have been impossible.

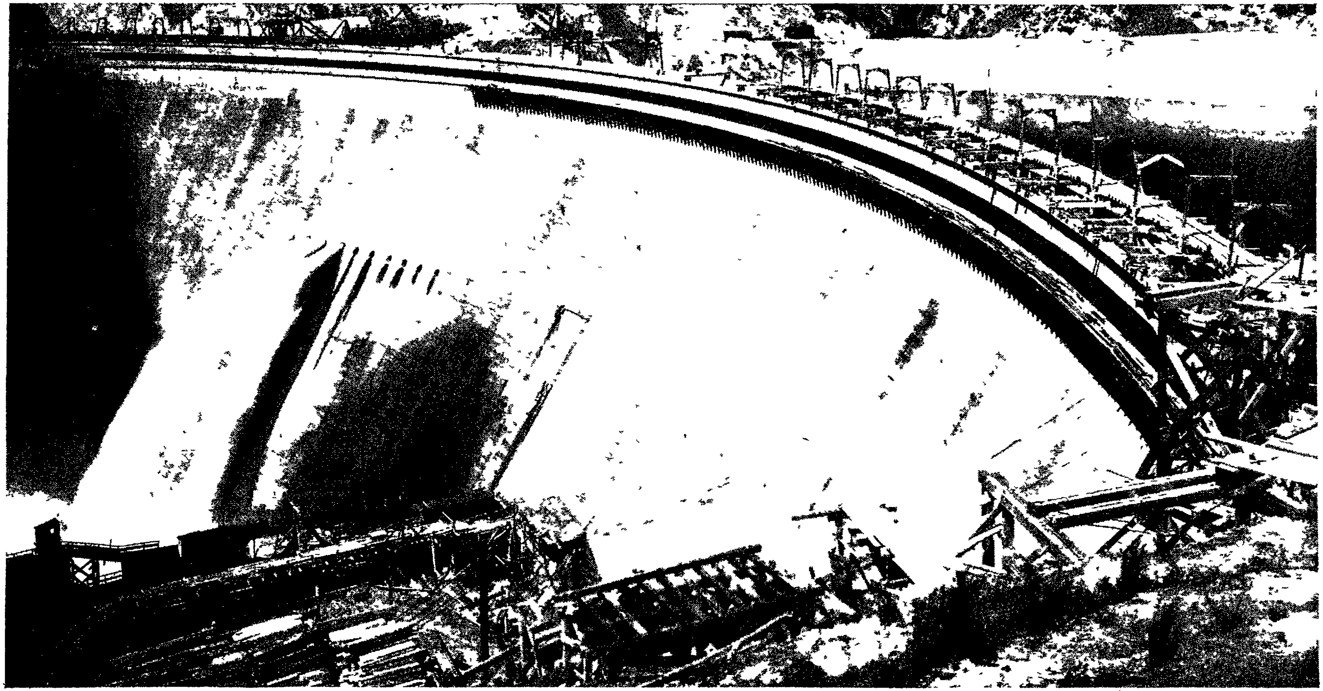
Table of Contents

VOLUME IX

---

* For page numbers, see foot of pages.

† For professional standing of authors, see list of Authors and Collaborators at front of volume.



VIEW OF NEARLY COMPLETED ARROW ROCK DAM ACROSS BOISE RIVER

This dam, which was dedicated on October 4, 1915, is the highest dam in the world, being 348 feet above bed rock and 250 feet above the river bed. Its length is 1060 feet.

Courtesy of U. S. Reclamation Service, Washington, D. C.

DAMS AND WEIRS

PART I

INTRODUCTION

1. Definitions. A dam may be defined as an impervious wall of masonry, concrete, earth, or loose rock which upholds a mass of water at its rear, while its face or lower side is free from the pressure of water to any appreciable extent. The waste water of the reservoir impounded by the dam is disposed of by means of a waste weir, or by a spillway clear of the work, or in rare cases, by sluice openings in the body of the dam.

Weirs, or overfall dams, although often confounded with bulkhead dams, differ from the latter in the following points, first, that the water overflows the crest, and second, that the tail water is formed below the dam. These two differences often modify the conditions of stress which are applicable in the design of dams proper, and consequently the subject of weirs demands separate treatment.

2. Classification. Dams and weirs may be classified as follows:

1. Gravity Dams
2. Gravity Overfalls, or Weirs
3. Arched Dams
4. Hollow Arch Buttress Dams
5. Hollow Slab Buttress Dams
6. Submerged Weirs
7. Open Dams, or Barrages

The subjects of earth, rock fill, and steel dams will not be treated in this article, as the matter has been already dealt with in other volumes. Graphical as well as analytical methods will be made use of, the former procedure being explained in detail as occasion demands.

GRAVITY DAMS

GENERAL DISCUSSION OF DAMS

A gravity dam is one in which the pressure of the water is resisted by the weight or "gravity" of the dam itself.

3. Pressure of Water on Wall. The hydrostatic pressure of the water impounded by a wall or dam may be graphically represented by the area of a triangle with its apex at the surface and its base drawn normal to the back line of the dam, which base is equal or proportionate to the vertical depth. When the back of the wall is vertical, as in Fig. 1, the area of this pressure triangle will be $\frac{H^2}{2}$ H being the vertical height. When, as in Fig. 2, the back is inclined, this area will be $\frac{H'H}{2}$, H' being the inclined length of the exposed surface, which equals H cosec ϕ.

The actual pressure of water per unit length of dam is the above area multiplied by the unit weight of water. This unit

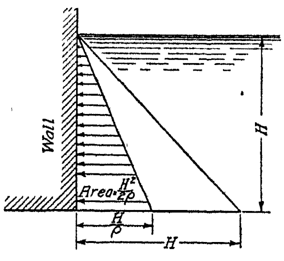


Fig 1. Water Pressure Area with Back of Dam Vertical

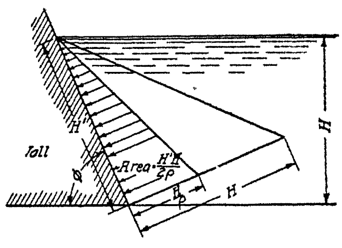


Fig. 2. Water Pressure Area with Back of Dam Inclined

weight is symbolized by w, which is 62.5 pounds, or $\frac{1}{32}$ short ton, or $\frac{1}{36}$ long ton, per cubic foot.

Unit Pressure. The pressure per square foot, or unit pressure on the wall at any point, is measured by the corresponding ordinate of the above triangle, drawn parallel to its base, and is evidently the same in both Figs. 1 and 2. The total pressure on the inclined back as represented by the triangle in Fig. 2 will, however, be greater than in Fig. 1.

4. Method for Graphical Calculations. For graphical calculations when forces of dissimilar unit weight or specific gravity are

engaged, as in the case of water and masonry, or earth and masonry, it is the usual practice to reduce them to one common denominator by making alterations in the areas of one or the other, the weight of the masonry being usually taken as a standard. This result is effected by making the bases of the triangles of water pressure equal, not to H, but to $\frac{H}{\rho}$, ρ (rho) being the sign of the specific gravity of the solid material in the wall. The triangle thus reduced will then represent a weight or area of masonry 1 unit thick, equivalent to that of water. This device enables the item of unit weight, which is $w \times \rho$ to be eliminated as a common factor from the forces engaged, i. e., of the water pressure and of the weight of the wall. The factor thus omitted has to be multiplied in again at the close of the graphical operation, only, however, in cases where actual pressures in tons or pounds are required to be known.

Value of ρ. The values ordinarily adopted for ρ, the specific gravity of masonry or concrete, are $2\frac{1}{4}$ and 2.4, i.e., equivalent to weights of 141 and 150 pounds, respectively, per cubic foot, while for brickwork 2 is a sufficiently large value. The value of $w\rho$ in the former case will be .069 ton and in the latter .075, or $\frac{3}{40}$ ton.

In some cases the actual value of ρ mounts as high as 2.5 and even 2.7, when heavy granite or basalt is the material employed.

The reduction thus made in the water pressure areas has further the convenience of reducing the space occupied by the diagram. The areas of the reduced triangles of water pressure in Figs. 1 and 2 are $\frac{H^2}{2\rho}$ and $\frac{H'H}{2\rho}$, respectively.

5. Conditions of "Middle Third" and Limiting Stress. Sections of gravity dams are designed on the well-known principle of the "middle third." This expression signifies that the profile of the wall must be such that the resultant pressure lines or centers of pressure due first to the weight of the dam considered alone, and second with the external water pressure in addition, must both fall within the middle third of the section on any horizontal base. These two conditions of stress are designated, Reservoir Empty (R.E.) and Reservoir Full (R.F.). The fulfillment of this condition insures the following requirement: *The maximum compressive ver-*

tical unit stress (s), *or reaction on the base of a dam, shall not exceed twice the mean compressive unit stress,* or, stated symbolically,

$$s \leqq 2s_1$$

Now the mean vertical compressive unit stress s_1 is the weight of the structure divided by its base length- i.e.,

$$s_1 = \frac{W}{b}$$

Hence, s, the maximum vertical unit pressure, should not exceed $\frac{2W}{b}$. Further comments on the distribution of the reaction on the base of a dam will be made in a later paragraph.

6. Compressive Stress Limit. A second condition imposed is that of the internal compressive stress limit, that is: *The maximum permissible compressive unit stress which is developed in the interior of the masonry of the dam, must not be exceeded.* This value can be experimentally found by crushing a cube of the material employed, and using a factor of safety of 6 or 8. Cement concrete will crush at about 2000 pounds per square inch, equivalent to 144 tons (of 2000 pounds) per square foot. The safe value of s would then be $\frac{144}{8} = 18$ tons per square foot. For ordinary lime concrete as employed in the East, the limit pressure adopted is generally 8 "long" tons, equivalent to 9 tons of 2000 pounds. Ten "long" tons, or 11.2 "short" tons is also a common value.

DESIGN OF DAMS

7. Theoretical Profile. The theoretically correct profile of a so-termed "low" masonry dam, i.e., one of such height that the limit stress is not attained under the conditions above outlined, is that of a right-angled triangle having its back toward the water vertical, and its apex at the water surface. It can be proved that the proper base width b of this triangle is expressed by the formula

$$b = \frac{H}{\sqrt{\rho}} \tag{1}$$

This profile, shown in Fig. 3, will be termed the "elementary triangular profile", as on it the design of all profiles of dams is more or less based. In this expression, H is the vertical height. The base

width of $\frac{H}{\sqrt{\rho}}$ insures the exact incidence of the vertical resultant (W) (R.E.) and of the inclined resultant R (R.F.) at the inner and outer edge, respectively, of the central third division of the base. The condition of the middle third is thus fulfilled in the most economical manner possible, a factor of safety of 2 against overturning is obtained, and further, the angle of inclination of the resultant R with regard to the base is usually such as to preclude danger of failure by sliding.

The fore slope or hypothenuse will be in the ratio $1:\sqrt{\rho}$ which, when $\rho=2\frac{1}{4}$, will equal 2:3, a slope very commonly adopted,

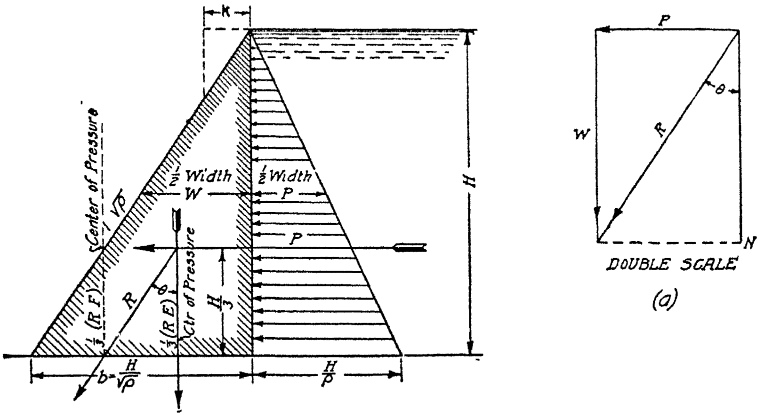


Fig 3. Elementary Triangular Profile for "Low" Masonry Dam

and with $\rho=2.4$ the ratio will be 1:1.549. The area of the elementary triangle is $\frac{H^2}{2\sqrt{\rho}}$ while, as we have seen, that of the water pressure is $\frac{H^2}{2\rho}$. θ is the vertical angle between W and R, and

$$\sec \theta = \frac{\sqrt{\rho+1}}{\sqrt{\rho}} = 1.187 \text{ with } \rho=2.4.$$

In Fig. 3 the resultant pressure lines are drawn to intersect the base so as to afford ocular proof of the stability of the section under the postulated conditions.

Graphical Method. The graphical procedure will now be briefly explained, and also in the future as fresh developments arise, for

the benefit of those who are imperfectly acquainted with this valuable labor-saving method.

There are two forces engaged, P the horizontal, or, it may be P_1, the inclined water pressure acting through the center of gravity of its area normal to the back of the wall, and W the weight or area of the wall. Of these two forces the item $w\rho$, or unit weight, has already been eliminated as a common factor, leaving the pressures represented by superficial areas. As, however, the height H is also common to both triangles, this can likewise be eliminated. The forces may then be represented simply by the half widths of the triangular areas by which means all figuring and scaling may be avoided.

First, a force polygon has to be constructed. In Fig. 3a, P is first drawn horizontally to designate the water pressure, its length being made equal to the half width of its pressure area in Fig. 3. From the extremity of P, the load line W is drawn vertically, equal to the half width of the elementary triangular profile, then the closing line R according to the law of the triangle of forces will represent the resultant in magnitude and direction. Second, the lines of actual pressures reciprocal to those on the force polygon will have to be transferred to the profile. The incidence of the resultant water pressure on the back is that of a line drawn through the c.g. of the area of pressure, parallel to its base, in this case, at $\frac{H}{3}$, or one-third the height of the water-pressure triangle, above the base. Its direction, like that of the base, is normal to the back, in this case horizontal, and if prolonged it will intersect the vertical force W, which in like manner acts through the center of gravity of the elementary profile of the wall. From this point of intersection the resultant R is drawn parallel to its reciprocal in Fig. 3a. Both W and R are continued until they cut the base line, and these points of intersection will be found to be exactly at the inner and outer edges of the middle third division of the base. It will be seen that when the reservoir is empty the center of pressure on the base is at the incidence of W, when full it is shifted to that of R.

Analytical Method. The same proof can be made analytically as follows: The weight of the two triangles W and P can be represented by their bases which are $\frac{H}{\sqrt{\rho}}$ and $\frac{H}{\rho}$, respectively. If moments

be taken about the outer edge of the middle third, the lever arm of the vertical force W is clearly $\frac{b}{3}$ or $\frac{H}{3\sqrt{\rho}}$ and that of P, the horizontal force, is the distance of the center of gravity of the triangle of water pressure above the base, viz, $\frac{H}{3}$. The equation will then stand

$$\left(\frac{H}{\sqrt{\rho}}\times\frac{H}{3\sqrt{\rho}}\right)-\left(\frac{H}{\rho}\times\frac{H}{3}\right)=0$$

or

$$\frac{H^2}{3\rho}-\frac{H^2}{3\rho}=0$$

If the actual values of R and of W were required, their measured or calculated lengths would have to be multiplied by H and by $w\rho$ in order to convert them to tons, pounds, or kilograms, as may be required. In many, in fact most, cases actual pressures are not required to be known, only the position of the centers of pressures in the profile.

Thus a line of pressures can be traced through a profile giving the positions of the centers of pressure without the necessity of converting the measured lengths into actual quantities. In the elementary triangle, Fig. 3, the value of the vertical resultant W is $\frac{H^2 w\sqrt{\rho}}{2}$. That of R required in the older methods of calculation is $\frac{H^2 w\sqrt{\rho+1}}{2}$. The following values relative to ρ will be found useful.

ρ or Specific Gravity	$\sqrt{\rho}$	$\frac{1}{\sqrt{\rho}}$	$\frac{1}{\rho}$	Pounds per Cubic Foot	Tons per Cubic Foot
2	1 414	71	.5	125	.0625
$2\frac{1}{4}$	1 5	$\frac{2}{3}$	$\frac{4}{9}$	141	07
2 4	1 55	645	.417	150	$.075=\frac{3}{40}$
2 5	1 58	633	4	156	.078
2 7	1 643	.609	.37	168 7	.084

Profile with Back Inclined. If the elementary profile be canted forward so that its back is inclined to the vertical, it will be found that the incidence of R will fall outside the middle third while that of W will be inside. The base will, therefore, have to be increased above $\frac{H}{\sqrt{\rho}}$.

When the back is overhanging, on the other hand, R will fall inside and W outside the middle third. The vertically backed section is consequently the most economical.

8. **Practical Profile.** In actual practice a dam profile must be provided with a crest of definite width, and not terminate in the apex of a triangle. The upper part of a dam is subjected to indefinite but considerable stresses of an abnormal character, due to extreme changes in temperature, consequently a solid crest is a necessity. The imposition of a rectangular crest, as shown in dotted lines on

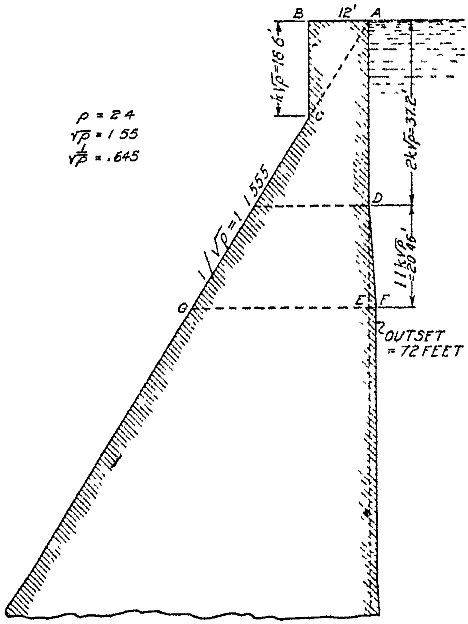


Fig 4 Practical Pentagonal Profile for "Low" Masonry Dam

Fig. 3, transforms the triangular profile into a pentagon. This has the effect of increasing the stability of the section (R.F.) so that the base width can be somewhat reduced, at the same time the vertical resultant W (R.E.), falls outside the middle third, but to so small an extent that this infringement of the imposed condition is often entirely neglected. In order to provide against this, a strip of material will have to be added to the back of the plain pentagonal profile. Fig. 4 is a diagram explanatory of these modifications. The dimensions of this added strip, as well as its position, can be

conveniently expressed in terms of (k) the crest width—i.e., AB in Fig. 4. The line of pressure (R.E.) will begin to leave the middle third at the depth AD, which is found by calculation which need not be produced here, to be $2k\sqrt{\rho}$. Below the point D, the divergence of the line of pressure will continue for a further depth DE, the point E, being close upon $3.1k\sqrt{\rho}$ below the crest, or $1.1k\sqrt{\rho}$ below D. Below point E, the line of pressure will no longer diverge outward, but will tend to regain its original position, consequently no further widening will be necessary, and the added

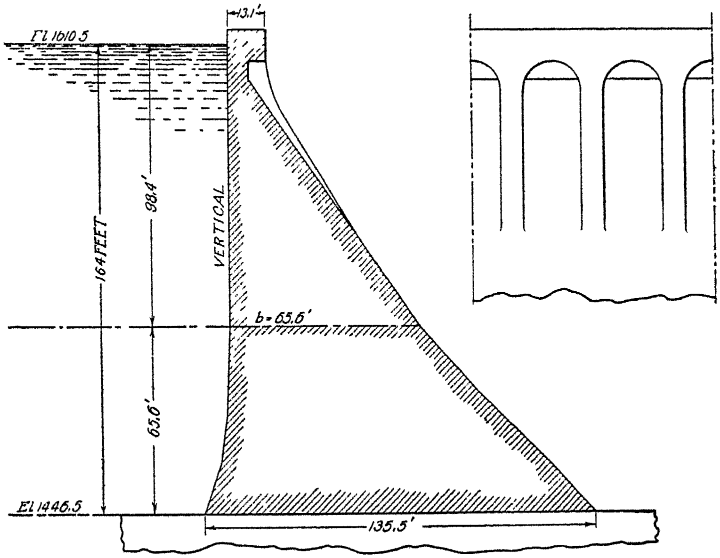


Fig 5. Profile of Chartrain Dam Showing Crest with Overhang

strip will be rectangular in form down to the base. The points D and F being joined, this portion of the back will be battered. The width of this added strip EF will be, with close approximation, $\frac{k}{16}$ or $.06k$.

9. Crest Width. The crest width of a dam should be proportioned to its actual height in case of a "low" dam, and in the case of a "high" dam to the limiting height—i.e., to that depth measured below the crest at which the maximum stress in the masonry is first

reached. Thus in "high" dams the upper part can always be of the same dimensions except where the requirements of cross communication necessitate a wider crest.

The effect of an abnormally wide crest can be modified by causing it to overhang the fore slope, this widening being carried by piers and arches. A good example of this construction occurs in the Chartrain dam, Fig. 5. The arches form a stiff but light finish to the dam and have a pleasing architectural effect. The same procedure, but in a less pronounced degree, is carried out in the Croton dam, Fig. 27.

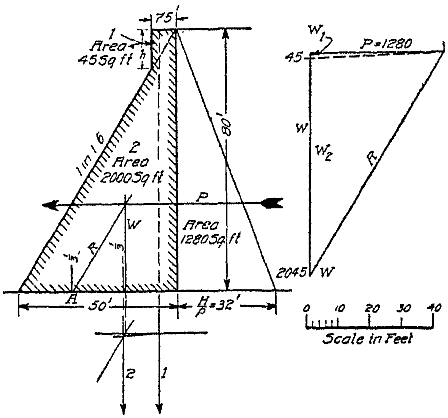


Fig 6 Pentagonal Profile—Back Vertical

The formula for crest width can be expressed either in terms of the limiting height H_l, or of the base b, where the limiting height is not attained, and a good proportion is given by the following empirical rule:

$$k=\sqrt{H_l} \qquad (2)$$

or

$$k=.15b \qquad (2a)$$

This latter formula makes the crest width a function of the specific gravity as well as of the height, which is theoretically sound.

10. Rear Widening. Where the rear widening of a "low" dam is neglected or where a uniform batter is substituted for the arrangement shown in Fig. 4, the profile will be pentagonal in outline. When the back is vertical the two triangles composing the body of the dam are similar. If the ratio existing between the crest (k) and the base (b), or $\frac{k}{b}$ be designated by r, then $k=br$, and h, the depth of the vertical side in Fig. 6, $=Hr$ and $k\times h=Hbr^2$.

In order to find what value the base width b should have, so that the center of pressure (R.F.) will fall exactly at the edge of the middle third, the moments of all the forces engaged will have to be taken about this point and equated to zero. The vertical forces

consist of W, the lower, and ~~W_1, the upper triangle;~~ the horizontal of P, the water pressure.

11. Method of Calculation. The pressures can be represented by the areas of the prisms involved, the triangle of water pressure being as usual reduced by dividing its base by ρ. A further elimination of common factors can be achieved by discarding $\frac{H}{2}$ which is common to all three forces, the area W_1 being represented by br^2 because the actual original value is $\frac{Hbr^2}{2}$. The forces then are W, represented by b; W_1 by br^2; and P by $\frac{H}{\rho}$; the actual value of the latter being $\frac{H^2}{2\rho}$. The lever arm distances of the c.g.'s of these three forces from A, the incidence of R, are as follows: of W, $\frac{b}{3}$, of W_1, $\frac{2}{3}(b-br)$, and of P, $\frac{H}{3}$. The equation will then stand, eliminating $\frac{1}{3}$,

$$b\times b+br^2(2b-2\times br)-\frac{H^2}{\rho}=0$$

or

$$b^2(1+2r^2-2r^3)-\frac{H^2}{\rho}=0$$

whence

$$b=\frac{H}{\sqrt{\rho}}\times\frac{1}{\sqrt{1+2r^2-2r^3}} \qquad (3)$$

The value of b thus obtained will prove a useful guide in deciding the base width even when the back of the wall is not vertical, as only a small increase will be needed to allow for the altered profile. When $\frac{k}{b}$ or $r=.15$ the reducing coefficient works out to $\frac{1}{1.019}$, the reciprocal of which is .981. Thus with a profile 80 feet high with $\rho=2.5$ and $r=.15$, the base width of the pentagonal profile will be $b=\frac{80}{\sqrt{2.5}}\times.981$ $=49.64$ feet; the decrease in base width below that of the elementary profile without crest will be $50.60-49.64=0.96$ feet. The crest width will be $49.64\times.15=7.45$ feet. In actual practice, the dimensions would be in round numbers, 50 feet base and $7\frac{1}{2}$ feet crest

width as made on Fig. 6. The face of the profile in Fig. 6 is made by joining the toe of the base with the apex of the triangle of water pressure.

Graphical Process. The graphical processes of finding the incidences of W and of R on the base are self-explanatory and are shown on Fig. 6. The profile is divided into two triangular areas, (1), 45 square feet and (2), 2000 square feet. The two final resultants fall almost exactly at the middle third boundaries, W, as might be conjectured, a trifle outside. Areas are taken instead of $\frac{1}{2}$ widths, owing to H not being a common factor.

Analytical Process. The analytical process of taking moments about the heel is shown below:

	Area	Lever Arm	Moment
(1)	45	$\frac{7.5\times 2}{3}$	225
(2)	2000	$\frac{50}{3}$	33333
W	2045		33558

The value 33,558, which is the total moment of parts equals the moment of the whole about the same point or

$$2045\times x=33558$$

$\therefore$

$$x=16.41 \text{ feet}$$

The incidence of W is therefore $\frac{50}{3}-16.41=.26$ ft. outside the middle third. To find that of R relative to the heel, the distance (see section 17) between W and R is $\frac{PH}{3W}=\frac{1280\times 80}{3\times 2045}=\frac{102400}{6135}=16.69$. The distance of R from the heel is therefore $16.69+16.41=33.10$ ft. The $\frac{2}{3}$ point is 33.33 feet distant, consequently the incidence of R is .23 foot within the middle third.

If the base and crest had been made of the exact dimensions deduced from the formula, the incidence of R would be exactly at the $\frac{2}{3}$ point while W would fall slightly outside the one-third point.

12. Variation of Height. The height of a dam is seldom uniform throughout; it must vary with the irregularities of the river bed, so that the maximum section extends for a short length only, while the remainder is of varying height. This situation will affect the relationship between the crest width and the height, and also the base width. To be consistent, the former should vary in width in proportion to the height. This, however, is hardly practicable, consequently the width of the crest should be based more on the average than on the maximum height, and could be made wider wherever a dip occurs in the foundation level.

13. High and Wide Crest. In case of a very high as well as wide crest, i.e., one carried much higher than the apex of the triangle of water pressure, it is not desirable to reduce the base width much below that of the elementary triangle. The excess of material in the upper quarter of a "low" dam can be reduced by manipulating the fore slope. This latter, which is drawn upward from the toe of the base, in Fig. 7, can be aligned in three directions. First, by a line terminating at the apex of the triangle of water pressure; second, it can be made parallel to that of the elementary profile, that is, it can be given an inclination of $\frac{1}{\sqrt{\rho}}$ to the vertical, and third, the slope or batter can be made flatter than the last. This latter disposition is only suitable with an abnormally high and wide crest and is practically carried out in the Chartrain dam, Fig. 5, where the base is not reduced at all below $\frac{H}{\sqrt{\rho}}$.

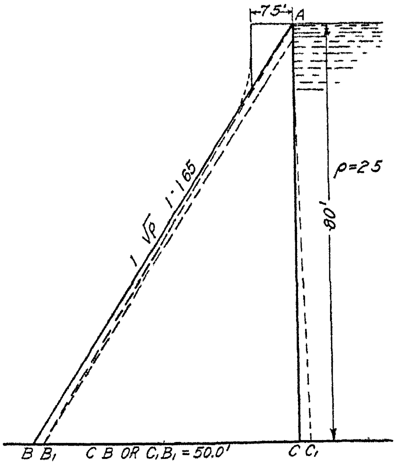


Fig 7. Profile Showing Different Disposition of Fore Slope

Reduction to any large extent, of the neck of the profile thus effected is, however, not to be commended, as the upper quarter of a dam is exposed to severe though indeterminate stresses due to

changes of temperature, wind pressure, etc., and also probably to masses of ice put in motion by the wind. The Cross River dam, to be illustrated later, as well as the Ashokan dam, are examples of an abnormally thick upper quarter being provided on account of ice. Whatever disposition of the fore slope is adopted, the profile should be tested graphically or analytically, the line of pressure, if necessary, being drawn through the profile, as will later be explained.

From the above remarks it will be gathered that the design of the section of a dam down to the limiting depth can be drawn by a few lines based on the elementary profile which, if necessary, can be modified by applying the test of ascertaining the exact position of the centers of pressure on the base. If the incidence of these resultants falls at or close within the edge of the middle third division of the base, the section can be pronounced satisfactory; if otherwise, it can easily be altered to produce the desired result.

Freeboard. The crest has to be raised above actual full reservoir level by an extent equal to the calculated depth of water passing over the waste weir or through the spillway, as the case may be. This extra freeboard, which adds considerably to the cost of a work, particularly when the dam is of great length and connected with long embankments, can be avoided by the adoption of automatic waste gates by which means full reservoir level and high flood level are merged into one.

In addition to the above, allowance is made for wave action, the height of which is obtained by the following formula:

$$h_w = 1.5\sqrt{F} + (2.5 - \sqrt[4]{F}) \qquad (4)$$

In this expression F is the "fetch", or longest line of exposure of the water surface to wind action in miles. Thus if $F = 4$ miles, the extra height required over and above maximum flood level will be $(1.5 \times 2) + (2.5 - 1.4) = 4.1$ feet. If $F = 10$ miles, h_w will work out to $5\frac{1}{2}$ feet. The apex of the triangle of water pressure must be placed at this higher level; the crest, however, is frequently raised still higher, so as to prevent the possibility of water washing over it.

14. Example. The working out of an actual example under assumed conditions will now be given by both graphical and analytical methods. Fig. 8 represents a profile 50 feet in height with crest level corresponding with the apex of the triangle of water pressure.

The assumed value of ρ is $2\frac{1}{4}$. The outline is nearly pentagonal, the crest width is made $.15b$ and the base width is the full $\frac{H}{\sqrt{\rho}}=\frac{2}{3}\times 50=33.3$ feet, the crest width is thus 5 feet. The back slope is carried down vertically to the point e, a distance of 8 feet, and from here on, it is given a batter of 1 in 50. The outset at the heel beyond the axis of the dam, which is a vertical line drawn through the rear

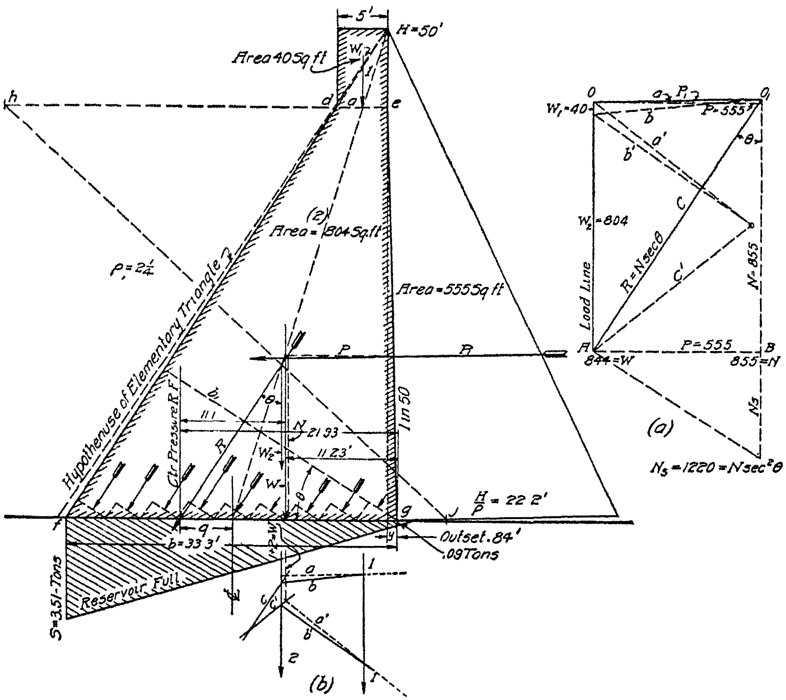


Fig. 8. Diagram Showing Suitable Profile for 50-Foot Dam

edge of the crest is therefore .84 foot. The toe is set in the same extent that the heel is set out. The face line of the body is formed by a line joining the toe with the apex of the water-pressure triangle. If the face line were drawn parallel to the hypothenuse of the elementary triangle, i.e., to a slope of $1:\sqrt{\rho}$, it would cut off too much material, the area of the wall being then but very little in excess of that of the elementary triangle, which, of course, is a minimum quantity. As will be seen later, the analysis of the section will show that the adopted base width could have been reduced below what

has been provided, to an extent somewhat in excess of that given in formula (3).

15. Graphical Method. The graphical procedure of drawing the resultant lines W (R.E.) and R (R.F.) to their intersection of the base presents a few differences, from that described in section 7, page 6, with regard to Fig. 3. Here the profile is necessarily divided into two parts, the rectangular crest and the trapezoidal body. As the three areas (1), (2), and P_1, are not of equal height, the item H cannot be eliminated as a common factor, consequently the forces will have to be represented as in Fig. 6 by their actual superficial areas, not by the half width of these areas as was previously the case. In Fig. 8a the vertical load line consists of the areas 1 and 2 totaling 844 square feet, which form W. The water pressure P_1 is the area of the inclined triangle whose base is $\frac{H}{\rho}$. This is best set out graphically in the force polygon by the horizontal line P, made equal to the horizontal water pressure, which is $\frac{H^2}{2\rho}=\frac{2500\times 2}{9}=555$ square feet. The water-pressure area strictly consists of two parts corresponding in depth to (1) and (2) as the upper part is vertical, not inclined, but the difference is so slight as to be inappreciable, and so the area of water pressure is considered as it would be if the back of the wall were in one inclined plane. In Fig. 8 the line P_1 normal to the back of the wall is drawn from the point of origin O and it is cut off by a vertical through the extremity of the horizontal line P. This intercepted length $O\,O_1$ is clearly the representative value of the resultant water pressure, and the line joining this point with the base of the load line W is R, the resultant of W and of P_1. If a horizontal line AB be drawn from the lower end of the load line W it will cut off an intercept (N) from a vertical drawn through the termination of P_1. This line $AB=P$, and N is the vertical component of R, the latter being the resultant of W and P_1 as well as of N and P. When the back is vertical, N and W are naturally identical in value, their difference being the weight of water overlying the inclined rear slope.

The further procedure consists in drawing the reciprocals of the three forces P_1, W, and R on the profile. The first step consists in finding the centers of gravity of the vertical forces 1 and 2 in which

the hexagonal profile is divided. That of (1) lies clearly in the middle of the rectangle whose base is *de*. The lower division (2) is a trapezoid. The center of gravity of a trapezoid is best found by the following extremely simple graphical process. From *d* draw *dh* horizontally equal to the base of the trapezoid *fg* and from *g*, *gj* is set off equal to *de;* join *hj*, then its intersection with the middle line of the trapezoid gives the exact position of its center of gravity. Thus a few lines effect graphically what would involve considerable calculation by analytical methods, as will be shown later.

The next step is to find the combined c. g. of the two parallel and vertical forces 1 and 2. To effect this for any number of parallel or non-parallel forces, two diagrams are required, first, a so-termed force and ray polygon and, second, its reciprocal, the force and chord, or funicular polygon. The load line in Fig. 8a can be utilized in the former of these figures. First, a point of origin or nucleus of rays must be taken. Its position can be anywhere relative to the load line, a central position on either side being the best. The point O_1, which is the real origin of the force polygon at the extremity of P_1 can be adopted as nucleus and often is so utilized, in which case the force line P_1 and R can be used as rays, only one additional ray being required. For the sake of illustration, both positions for nucleus have been adopted, thus forming two force and ray polygons, both based on the same load line, and two funicular polygons, the resultants of which are identical. The force and ray polygon is formed by connecting all the points on the load line with the nucleus as shown by the dotted line *a*, *b*, and *c*, and *a′*, *b′*, and *c′*. Among the former, *a* and *c* are the force lines P_1 and R, the third, *b*, joins the termination of force (1) on the load line with the nucleus. These lines *a*, *b*, *c*, are the rays of the polygon. Having formed the force and ray diagram, in order to construct the reciprocal funicular polygon 8*b* the force lines (1) and (2) on the profile Fig. 8 are continued down below the figure. Then a line marked (*a*) is drawn anywhere right through (1) parallel to the ray *a*, from its intersection with the force (1), the chord (*b*) is drawn parallel to the ray (*b*) in Fig. 8b meeting (2); through this latter intersection the third chord (*c*) is drawn backward parallel to its reciprocal the ray *c*. This latter is the closing line and its intersection with the initial line (*a*), gives the position of the c. g. of the two forces.

A vertical line through this center of pressure, which represents W, i.e., W_1+W_2, is continued on to the profile until it intersects the inclined force P_1 drawn through the center of gravity of the water pressure area. This intersection is the starting point of R, drawn parallel to its reciprocal on the force polygon 8*a*. This resultant intersects the base at a point within the middle third. R is the resultant "Reservoir Full", while W, the resultant of the vertical forces in the masonry wall, is the resultant "Reservoir Empty". The intersection of the latter is almost exactly at the inner edge of the middle third—thus the condition of the middle third is fulfilled. The question of induced pressure and its distribution on the base will be considered later.

The incidence of N, the vertical component "Reservoir Full", on the base is naturally not identical with that of W, the resultant "Reservoir Empty", unless the back of the wall is vertical. The line R is the resultant of both P_1 and W, and of P and N. If it be required to fix the position of N on the profile, a horizontal line should be drawn through the intersection of P_1 with the back of the wall. This will represent the horizontal component of the water pressure P_1, and it will intersect R, produced upward. Then a line drawn vertically through this latter point will represent N, the vertical component (Reservoir Full). The position of N is necessarily outside of W, consequently if N is made to fall at the inner edge of the middle third of the base, W must fall within the middle third. This fact will later be made use of when the design of the lower part of a "high" dam comes under consideration.

16. Analytical Method. The analytical method of ascertaining the positions of the incidences of W and of R on the base, which has just been graphically performed, will now be explained.

The first step is to find the positions of the centers of gravity of the rectangle and trapezoid of which the profile is composed, relative to some vertical plane, and then to equate the sum of the moments of those two forces about any fixed point on the base, with the moment of their sum.

The most convenient point in most cases is the heel of the base; this projects a distance (y) beyond the axis of the dam, which axis is a vertical line passing through the inner edge of the rectangular crest.

As the areas of the divisions, whether of the masonry wall or of the water-pressure triangle, are generally trapezoids, the following enumeration of various formulas, whereby the position of the c.g. of a trapezoid may be found either with regard to a horizontal or to a vertical plane, will be found of practical utility. In Fig. 9, if the depth of the figure between the parallel sides be termed H, and that of the truncated portion of the triangle of which the trapezoid is a portion be termed d, and h be the vertical height of the c.g. above the base, then

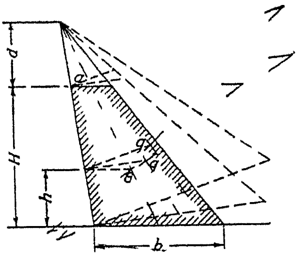


Fig 9 Diagram Showing Centers of Gravity of Water Pressure Trapezoids

$$h=\frac{H}{3}\times\frac{H+3d}{H+2d} \tag{5}$$

Thus, in Fig. 9, $H=13$ and $d=6$ feet, then

$$h=\frac{13}{3}\left(\frac{13+18}{13+12}\right)=5.37 \text{ feet}$$

If the base of the triangle and trapezoid with it be increased or decreased in length, the value of h will not be thereby affected, as it is dependent only on H and d, which values are not altered. If, however, the base of the triangle be inclined, as shown by the

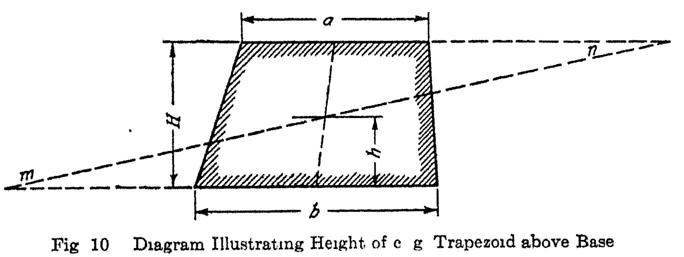


Fig 10 Diagram Illustrating Height of c g Trapezoid above Base

dotted lines in Fig. 9, the center of gravity of the trapezoid will be higher than before, but a line drawn parallel to the inclined base through g, the c. g. will always intersect the upright side of the trapezoid at the same point, viz, one which is h feet distant vertically above the horizontal base.

The value of h can also be obtained in terms of a and b, the two parallel sides of the trapezoid, and is

$$h=\frac{H}{3}\left(\frac{b+2a}{a+b}\right) \tag{6}$$

For example, in Fig. 10, $H=12$, $a=10$, and $b=16$, then

$$h=\frac{12}{3}\left(\frac{16+20}{10+16}\right)=5.54 \text{ feet}$$

If the horizontal distance of the c.g. of a trapezoid from a vertical plane is required, as, for example, that of the trapezoid in Fig. 8, the following is explanatory of the working. As shown in Fig. 11, this area can be considered as divided into two triangles, the weight of each of which is equivalent to that of three equal weights placed at its angles; each weight can thus be represented by one-third of the area of the triangle in question, or by $\frac{aH}{6}$ and $\frac{bH}{6}$, respectively, H being the vertical depth of the trapezoid. Let y be the projection of the lower corner A beyond that of the upper one B. Then by equating the sum of the moments of the corner weights about the point A with the moment of their sum, the distance (x) of the c.g. of the whole trapezoid from A will be obtained as follows:

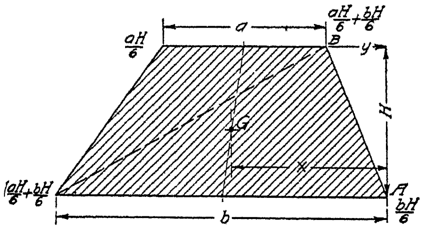


Fig 11 Method of Finding Distance of Center of Gravity of a Trapezoid from Heel

$$\left(\frac{a+b}{2}\right)Hx=\frac{H}{6}\left[b(a+b)+a(a+y)+y(a+b)\right]$$

$$\therefore \quad x=\frac{1}{3}\left[(b+y)+a\left(\frac{a+y}{a+b}\right)\right] \tag{7}$$

where $y=0$, the formula becomes

$$x=\frac{1}{3}\left(b+\frac{a^2}{a+b}\right) \tag{7a}$$

For example, in Fig. 8, a or $de=5$ feet, $b=33.3$, and $y=.84$, whence

$$x=\frac{1}{3}\left(34.14+\frac{29.2}{38.3}\right)=11.63 \text{ feet}$$

The similar properties of a triangle with a horizontal base, as in Fig. 12, may well be given here and are obtained in the same way by taking moments about A, thus

$$\frac{bh}{2}\times x=\frac{bh}{6}\ (b+y)$$

$\therefore$

$$x=\frac{b+y}{3} \qquad \textbf{(8)}$$

In Fig. 12, $b=14$ feet, $y=8$ feet, and $h=10$ feet, then

$$x=\frac{14+8}{3}=7\tfrac{1}{3}\text{ feet}$$

Reverting to Fig. 8, the position of the incidence of W on the base is obtained by taking moments about the heel g of the base as follows: Here W is the area of the whole profile, equal, as we have seen, to 844 sq. ft. The area of the upper component (1) is 40 sq. ft. and of (2) 804.

The lever arm of W is by hypothesis x, that of (1) is $2.5+.84=3.34$ feet, that of (2) by formula (7) has already been shown to be 11.63 feet. Hence, as the moment of the whole is equal to the sum of the moments of the parts, the equation will become

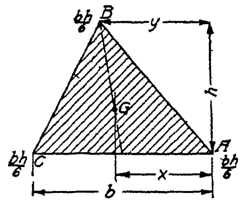


Fig 12 Method of Finding Center of Gravity of a Triangular Profile

$$844x=40\times 3.34+804\times 11.63=9484.1$$

$\therefore$

$$x=11.23\text{ feet}$$

This fixes the position of the incidence of W relative to the heel. The position of the inner third point is $\frac{b}{3}$, or $\frac{33.3}{3}$ from the heel. The incidence of W is therefore $11.23-11.10=.13$ foot within the middle third, which complies with the stipulated proviso.

The next step is to find the position of R relative to the heel of the base. As in graphical methods, only horizontal and vertical forces are considered; the water-pressure area is split into two parts, one, P the horizontal component, the value of which is $\frac{H^2}{2\rho}$, or 555 feet, and w_3 the reduced area of water overlying the rear projection of the back. The latter area is a trapezoid of which the upper side

(a) is 8 feet long and b, the lower side, 50 feet, the depth being .84 foot, hence the distance of its c.g. inside the heel of the base will be by formula (6), $\frac{.84}{3}\frac{(50+16)}{58}=.32$ foot. Its actual area is $\frac{8+50}{2}\times .84=24.4$ feet; this has to be divided by ρ or $2\frac{1}{4}$ to reduce it to a masonry base. The reduced area will then be 10.8 square feet, nearly. The distance of the incidence of W from the heel of the base has already been determined to be 11.23 ft. and that of w_3 being .32 ft., the distance of the c.g. of the latter from W will be $11.23-.32=10.9$, nearly. If the distance between the incidences of W and R be termed x, the equation of moments about the incidence of R, will stand thus:

$$P\frac{H}{3}=Wx+w_3(x+10.9)$$

or

$$555\times\frac{50}{3}=844x+10.8x+117.83$$

i.e.

$$x=\frac{9132.2}{854.8}=10.7 \text{ ft., nearly}$$

R is therefore $10.7+11.23=21.93$ ft. distant from the heel. The $\frac{2}{3}$ point being 22.2 ft. from the same point, R falls .3 ft. (nearly) within the middle third. This shows that a small reduction in the area of the profile could be effected.

17. Vertical Component. If the position of N, the vertical component of R and P_1, is required, as is sometimes the case, it is obtained by the equation $N\times x=(W\times 11.23)+(w_3\times .32)$, x being the distance from the heel of the base. Or in figures,

$$854.8x=(844\times 11.23)+(10.8\times .32)$$

$\therefore$

$$x=11.1 \text{ feet}$$

The incidence of N is, therefore, in this case, exactly on the limit of the middle third. This of course does not affect the condition of middle third, which refers to the resultant W (R.E.) not to the component N (R.F.) but, as will be seen later, when the lower part of a high dam comes to be designed, one condition commonly imposed is, that the vertical component N must fall at the inner edge of the middle third, in which case W will necessarily fall inside thereof.

It may here be noted that the space between the location of N and R, which will be designated (f), is $\frac{PH}{3N}$ because if moments are taken about the incidence of R, then $Nf=\frac{PH}{3}$; therefore $f=\frac{PH}{3N}$. The actual value of W in tons of 2000 pounds will be the superficial area, or 844 square feet multiplied by the eliminated unit weight, i.e., by wp, viz, $\frac{844\times 9}{32\times 4}=59.3$ tons, as $w=\frac{1}{32}$ ton. That of the inclined force R, is obtained from the triangle of forces PNR in which R, being the hypothenuse $=\sqrt{N^2+P^2}$. Here $N=855$ square feet, equivalent to 60 tons, nearly, and $P=555$ feet, equivalent to 39 tons, whence $R=\sqrt{60^2+39^2}=71.5$ tons.

18. Pressure Distribution. In the design of the section of a dam, pier, or retaining wall, the distribution of pressure on a plane in the section and the relations existing between maximum unit stress, symbolized by (s), and mean or average unit stress (s_1) will now be considered. The mean unit stress on any plane is that which acts at its center point and is in amount the resultant stress acting on the plane (the incidence of which may be at any point) divided by the width of the lamina acted on. Thus in Figs. 3 or 8 take the resultant W. This acts on the horizontal base and its mean unit stress s_1 will be $\frac{W}{b}$. In the same way, with regard to N, the vertical component of R the mean unit stress produced by it on the horizontal base will be $\frac{N}{b}$. The maximum unit stress occurs at that extremity of the base nearest to the force in question which is R. Thus the maximum unit stress due to W is at the heel while that due to a combination of P and N acting at the incidence of R is at the toe of the base b. It is evident that the nearer the incidence of the center of pressure is to the center point the less is the maximum stress developed at the outer edge of the section, until the center of pressure is actually situated at the center point itself. The maximum pressure at the outer part of the section then equals the average and is thus at a minimum value. The relation between maximum and mean unit stress or reaction is expressed in the fol-

lowing formula in which it is assumed that any tension at the heel can be cared for by the adhesion of the cementing material or of reinforcement anchored down:

$$s=s_1\left(1+\frac{6q}{b}\right) \qquad (9)$$

or, letting m equal the expression in brackets,

$$s=ms_1 \qquad \mathbf{(9a)}$$

In formula (**9a**), q is the distance between the center point of the base and the center of pressure or incidence of whatever resultant pressure is under consideration, and s_1 is the mean stress, or the resultant pressure divided by the base.

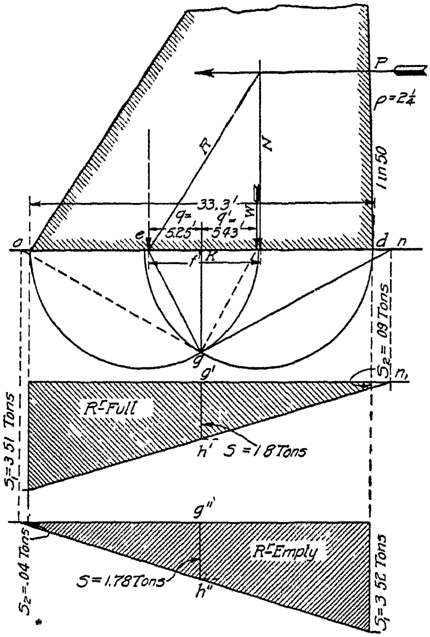


Fig. 13. Diagram Showing Pressure Distribution on a Dam with Reservoir Empty and Reservoir Full

In Fig. 8 as explained in section 16, the incidence of R, i.e., the center of pressure (R.F.), falls .3 ft. within the middle third of the base, consequently the value of q will be $\frac{b}{6}-.3=\frac{33.3}{6}-.3=5.25$ ft., and in formula (**9a**) $m=1+\frac{6q}{b}=1+\frac{31.5}{33.3}=1.95$. The maximum reaction at the toe always designated by $s=\frac{mN}{b}=\frac{1.95\times 60}{33.3}=3.51$ tons per sq. ft. For the reaction (R.F.) at the heel, $m=1-.95=.05$, and $s_2=\frac{.05\times 60}{33.3}=.09$ tons. The distribution of pressure due to the vertical component of R is shown hatched in Fig. 8 as well as in Fig. 13.

From formula (9) the facts already stated are patent. When the incidence of the resultant force is at the center of the base,

$q=0$, consequently $m=1$ and $s=s_1$, that is, the maximum is equal to the mean; when at one of the third points, $q=\frac{b}{6}$, $m=2$, and $s=2s_1$; when at the toe, $m=4$, and $s=4s_1$, or $4\frac{W}{b}$.

If the material in the dam is incapable of caring for tensile strain, the maximum vertical compression, or s, obtained by formula (9) will not apply. Formula (**24**), section 86, should be used whenever R falls outside the middle third.

In designing sections it is often necessary to maneuver the incidence of the resultant stress to a point as close as possible to the center of the base in order to reduce the maximum stress to the least possible value, which is that of the mean stress. The condition of the middle third, insures that the maximum stress cannot exceed twice the mean, and may be less, and besides insures the absence of tensile stress at the base.

19. Graphical Method for Distribution of Pressure. The graphical method of ascertaining the distribution of pressure on the base of a masonry wall, which has already been dealt with analytically, is exhibited in Fig. 13, which is a reproduction of the base of Fig. 8. The procedure is as follows: Two semicircles are struck on the base line, having their centers at the third division points and their radii equal to $\frac{b}{3}$. From the point marked e, that of the incidence of R, the line eg is drawn to g, the point of intersection of the two semicircles. Again from g a line gn is set off at right angles to eg cutting the base or its continuation at a point n. This point is termed the antipole of e, or the neutral point at which pressure is *nil* in either sense—compressive or tensile. Below and clear of the profile a projection of the base is now made, and from g a perpendicular is let fall, cutting the new base in g' while, if the line be continued upward, it will intersect the base at K. This latter point will, by the construction, be the center point of the base. The line Kg is continued through g' to h', $g'h'$ being made equal to the mean unit pressure, $=1.8$ tons. A perpendicular is let fall from n cutting the new base line at n_1; the points n_1 and h' are then joined and the line continued until it intersects another perpendicular let fall from the toe of the base. A third perpendicular is drawn

from the heel of the base, cutting off a corner of the triangle. The hatched trapezoid enclosed between the last two lines represents the distribution of pressure on the base. The maximum stress will scale close upon 3.51 and the minimum .09 tons. If W be considered, $s=\frac{W}{b}=\frac{59.3}{33.3}=1.78$ tons, the maximum stress at the heel will be 3.52 and the minimum .04, at the toe.

20. Examples to Illustrate Pressure Distribution. In Fig. 14 is illustrated the distribution of pressure on the base, due to the incidence of R, first, at the toe of the base, second, at the two-third point, third, at the center, and fourth, at an intermediate position. In the first case (R_1), it will be seen that the neutral point n_1 falls at the first third point. Thus two-thirds of the base is in compression and one-third in tension, the maximum in either case being proportional to the relative distance of the neutral point from the toe and heel of the base, the compression at the toe being four times, while the tension at the heel is twice the mean stress. In the second case (R_2) intersects at the two-third point, and the consequent position of n is exactly at the heel. The whole base is thus in compression, and the maximum is double the mean. In the third case (R_3), the line gn is drawn at right angles to fg. The latter is vertical and gn will consequently be horizontal. The distance to n is thus infinite and the area of pressure becomes a rectangle with a uniform unit stress s. In the fourth case (R_4), the neutral point lies well outside the profile, consequently the whole is in compression, the condition approximating to that of R_3.

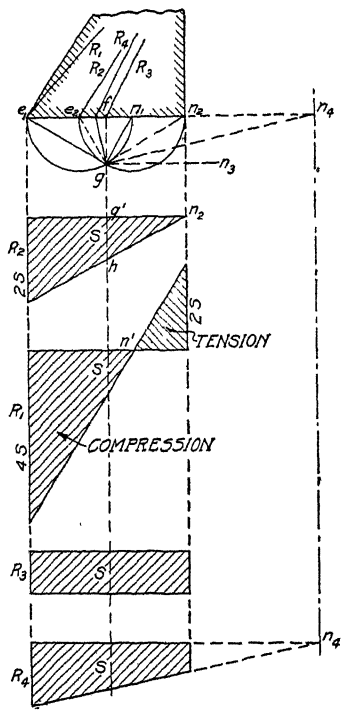


Fig 14 Pressure Distribution on Base of Dam under Various Conditions

21. Maximum Pressure Limit. The maximum pressure increases with the depth of the profile until a level is reached where the limit stress or highest admissible stress is arrived at. Down to this level the design of the section of a dam, as already shown, consists simply in a slight modification of the pentagonal profile with a vertical back, the base width varying between that of the elementary profile or $\frac{H}{\sqrt{\rho}}$, or its reduced value given in formula (3). Beyond this limiting depth, which is the base of the so-termed "low" dam, the pentagonal profile will have to be departed from and the base widened out on both sides.

22. Formulas for Maximum Stress. The maximum unit stress in the interior of a dam is not identical with (s), the maximum vertical unit reaction at the base, but is a function of s_1. In Fig. 8, a representative triangle of forces is shown composed of N the vertical force (R.F.), P the horizontal water pressure, and R the resultant of N and P; therefore $R=\sqrt{N^2+P^2}=$ also $N \sec \theta$. If the back were vertical, N and W would coincide and then $R=\sqrt{W^2+P^2}$. Various views have been current regarding the maximum internal stress in a dam. The hitherto most prevalent theory is based on the assumption, see Fig. 8, that the maximum unit stress

$$c=\frac{mR}{b}=m\frac{\sqrt{N^2+P^2}}{b}=\frac{mN}{b}\sec\theta \qquad \textbf{(10a)}$$

Another theory which still finds acceptance in Europe and in the East assumes that the maximum stress is developed on a plane normal to the direction of the resultant forces as illustrated by the stress lines on the base of Fig. 8. According to this, the mean stress due to R would not be $\frac{R}{b}$ but $\frac{R}{b_1}$, and the maximum stress will be $\frac{mR}{b_1}$. But $\frac{R}{b_1}=\frac{R\sec\theta}{b}$ and $R=N\sec\theta$, consequently the maximum unit stress would be

$$c=\frac{mN}{b}\sec^2\theta \qquad \textbf{(10b)}$$

Recent experiments on models have resulted in the formula for maximum internal unit stress being recast on an entirely different principle from the preceding. The forces in action are the maxi-

mum vertical unit force or reaction s combined with a horizontal shearing unit stress $s_s=\frac{P}{b}$. The shearing force is the horizontal water pressure, or $\frac{H^2w}{2\rho}$ symbolized by P, which is assumed to be equally resisted by each unit in the base of the dam; the unit shearing stress will thus be $\frac{P}{b}$. These forces being at right angles to each other, the status is that of a bar or column subject to compression in the direction of its length and also to a shear normal to its length. The combination of shear with compression produces an increased compressive stress, and also a tension in the material. The formula recently adopted for maximum unit compression is as follows:

$$c=\tfrac{1}{2}s+\sqrt{\tfrac{1}{4}s^2+s_s^2} \qquad \textbf{(10)}$$

In this $s=ms_1,=\frac{mN}{b}$. As before $s_s=\frac{P}{b}$, substituting we have

$$c=\frac{mN}{2b}+\sqrt{\frac{(mN)^2}{4b^2}+\frac{P^2}{b^2}}=\frac{mN+\sqrt{(mN)^2+4P^2}}{2b} \qquad \textbf{(10}_1\textbf{)}$$

When $m=2$, as is the case when the incidence of R is exactly at the outer boundary of the middle third

$$c=\frac{N+\sqrt{N^2+P^2}}{b}=\frac{N}{b}(1+\sec\theta) \qquad \textbf{(10}_2\textbf{)}$$

23. Application of All Three Formulas to Elementary Profile. In the case of elementary triangular profile which has a vertical back, $N=W$ and $\sec\theta=\frac{\sqrt{\rho+1}}{\sqrt{\rho}}$ (section 7, page 5) and $m=2$; then formula (**10**$_2$) becomes

$$c=\frac{W}{b}(1+\sec\theta)=\frac{W}{b}\left(1+\sqrt{\frac{\rho+1}{\rho}}\right)$$

Now

$$\frac{W}{b}=\frac{H^2}{2\sqrt{\rho}}\times\rho w\times\frac{\sqrt{\rho}}{H}=\frac{Hw\rho}{2}$$

$$c=\frac{Hw\rho}{2}\left(1+\sqrt{\frac{\rho+1}{\rho}}\right) \qquad \textbf{(11)}$$

Example.

Let H in elementary triangle $=150$ feet, $\rho=2.4$, $w\rho=\frac{3}{40}$ ton. When, according to (**11**), $c=\frac{150\times3}{2\times40}(1+1.187)=12.3$ tons per square foot.

Taking up formula (**10a**)

$$c=\frac{mW\sec\theta}{b}=\frac{2W\sec\theta}{b}$$

as above $\frac{W}{b}=\frac{Hw\rho}{2}$

$\therefore$

$$c=Hw\rho\sqrt{\frac{\rho+1}{\rho}}=Hw\sqrt{\rho}\sqrt{\rho+1} \qquad \textbf{(11a)}$$

Example with conditions as before

$$c=\frac{150\times1\times1.55\sqrt{3.4}}{32}=7.26\times1.84=13.3 \text{ tons}$$

With formula (**10b**), $c=\frac{2W\sec^2\theta}{b}$, or in terms of H,

$$c=Hw\rho\left(\frac{\rho+1}{\rho}\right)=Hw(\rho+1) \qquad \textbf{(11b)}$$

Therefore, with values as above,

$$c=\frac{150\times1\times3.4}{32}=15.9 \text{ tons}$$

From the above it is evident that formulas (**10b**) and (**11b**) give a very high value to c. Tested by this formula, high American dams appear to have maximum compressive unit stresses equal to 20 tons per square foot, whereas the actual value according to formula (**10**) is more like 14 tons. However, the stresses in the Assuan dam, the Periyar, and other Indian dams, as also French dams have been worked out from formula (**10b**) which is still in use.

24. Limiting Height by Three Formulas. The limiting height (H_l) of the elementary triangular profile forms a close guide to that obtaining in any trapezoidal section, consequently a formula will be given for each of the three cases in connection with formulas (**10**), (**10a**), and (**10b**). Referring to case (**10**), we have from formula (**11**)

$$c=\frac{Hw\rho}{2}\left(1+\sqrt{\frac{\rho+1}{\rho}}\right)$$

Whence H_l, the limiting height $=\dfrac{2c}{w\rho\left(1+\sqrt{\dfrac{\rho+1}{\rho}}\right)}$

Example.

With $c=16$ tons and $\rho=2.4$, H_l, the limit height of the elementary profile will be $\dfrac{2\times16\times33}{2.4\ (2.187)}=\dfrac{1024}{5.254}=195$ feet.

Referring to case (**10a**), we have from formula (**11a**)

$$c=Hw\ \sqrt{\rho}\sqrt{\rho+1}$$

$\therefore$

$$H_l=\frac{c}{w\sqrt{\rho}\sqrt{\rho+1}}$$

Example.

With data as above $H=\dfrac{16\times32}{1.55\times1.84}=180$ feet, nearly.

Referring to case (**10b**), we have from formula (**11b**)

$$c=Hw(\rho+1)$$

$\therefore$

$$H_l=\frac{c}{w(\rho+1)}$$

Example.

With same data $H_l=\dfrac{16\times32}{3.4}=\dfrac{512}{3.4}=150$ feet.

Thus the new formula (**10**) gives much the same results as that formerly in general use in the United States (**10a**), while in the more conservative formula (**10b**) the difference is marked.

25. Internal Shear and Tension. We have seen that the combination of compressive and shearing stresses in a dam (R.F.) produces an increased unit compression. It further develops an increase in the shearing stress and also a tensile stress. The three formulas are given below.

Compression as before

$$c=\frac{1}{2}s+\sqrt{\frac{s^2}{4}+s_s{}^2}\ \text{ or }\ \frac{mN+\sqrt{(mN)^2+4P^2}}{2b}\qquad \textbf{(10)}$$

Tension

$$t=\frac{1}{2}s-\sqrt{\frac{s^2}{4}+s_s{}^2}\ \text{ or }\ \frac{mN-\sqrt{(mN)^2+4P^2}}{2b}\qquad \textbf{(12)}$$

Shear

$$s_h=\sqrt{\frac{s^2}{4}+s_s{}^2} \qquad \text{or} \qquad \sqrt{\frac{(mN)^2+4P^2}{2b}} \qquad \textbf{(13)}$$

The tensile and shearing stresses are not of sufficient moment to require any special provision in the case of a gravity dam. The tension is greatest at the heel, diminishing toward the toe. This fact suggests that a projection of the heel backward would be of advantage. The direction (a) of c to the vertical is not that of R but is as follows:

$$\text{Tan } 2a=\frac{2s_s}{s}=\frac{2P}{b}\div\frac{mN}{b}=\frac{2P}{mN}$$

when $m=2$, $\tan 2a=\frac{P}{N}$. In Fig. 8, $P=555$ and $N=855$. $\therefore \tan 2a=\frac{555}{855}=.649$ whence $2a=33°\ 00'$ and $(a)=16°\ 30'$. The inclination of R to the vertical, or θ, is $33°\ 50'$, i.e., twice as large as that of c. The direction of t is at right angles to that of c, while that of s_h the shear, lies at 45° from the directions of either c or t.

26. Security against Failure by Sliding or Shear. Security against failure by sliding depends on the inclination of W to P, i.e., on the angle θ between W and R. Thus tan θ should be less than the angle of friction of masonry on masonry, or less than .7. This is the same as stating that the relation of W and P must be such that θ shall not be greater than 35°, or that the complement of θ be not less than 55°. The adoption of the middle third proviso generally insures this. With regard to sliding on the base, this can be further provided against by indentations in the base line or constructing it inclined upward from heel to toe.

27. Influence Lines. It is sometimes desirable for the purpose of demonstrating the correctness of a profile for tentative design, to trace the line of pressures corresponding to the two conditions of reservoir full and empty, through the profile of a dam. This is far better effected by the use of graphic statics.

There are two different systems of graphic construction that give identical results, which will now be explained and illustrated.

The first method, which is most commonly adopted, is exhibited in Fig. 15, which is the profile of a 100-foot high dam with specific gravity $2\frac{1}{4}$. It thus lies within the limiting depth, which for the elementary profile would be 190 feet.

The profile is pentagonal, with a vertical back, and has the full base width of the elementary profile, viz, $\frac{H}{\sqrt{\rho}}$ which in this case is $\frac{2}{3}\times100=66.7$ feet. The crest k is $\sqrt{H}=10$ feet wide. The water-pressure triangle has a base of $\frac{H}{\rho}$. The profile, as well as the water-pressure triangle, is divided into five equal laminas, numbered 1

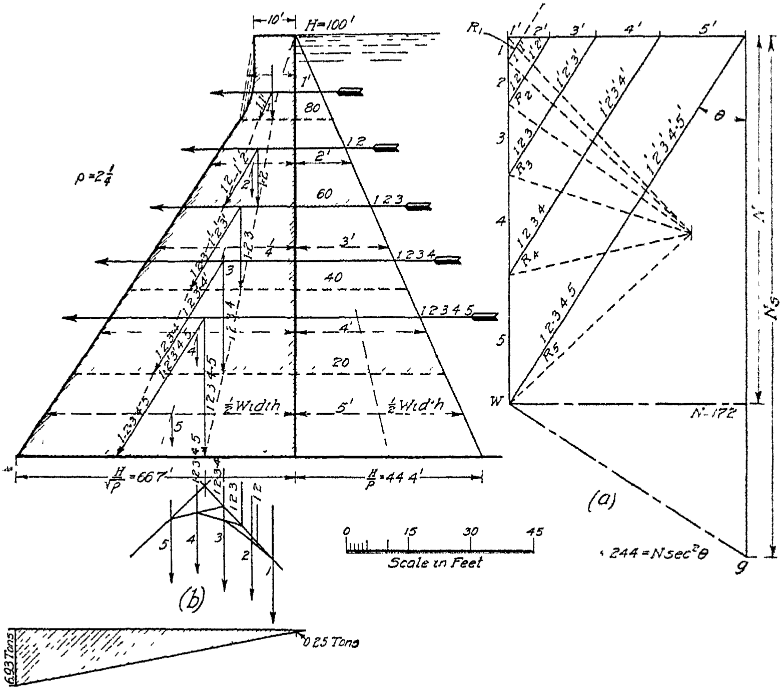


Fig 15. Graphical Construction for Tracing Lines of Pressure on Dam of Pentagonal Profile

to 5 in one case and 1′ to 5′ in the other. The depth of each lamina, which is $\frac{H}{5}$ is, therefore, a common factor and can be eliminated as well as the item of unit weight, viz, $w\rho$. The half widths of all these laminas will then correctly represent their areas and also their weights, reduced to one denomination, that of the masonry. In Fig. 15a a force polygon is formed. In the vertical load line the several half widths of the laminas 1 to 5 are first set off, and at

right angles to it the force line of water pressure is similarly set out with the half widths of the areas 1′ to 5′. Then the resultant lines of the combination of 1 with 1′, 1, 2, with 1′ 2′ and so on marked R_1 to R_5 are drawn. This completes the force polygon. The next step is to find the combinations of the vertical forces on the profile, viz, that of 1 and 2, 1, 2, and 3, etc. This, as usual, is effected by constructing a force and ray polygon, utilizing the load line in Fig. 15a for the purpose. Then the centers of gravity of the several individual areas 1 to 5 are found by the graphical process described in section 15, and verticals drawn through these points are projected below the profile. On these parallel force lines 1 to 5, the funicular polygon Fig. 15b is constructed, its chords being parallel to their reciprocal rays in Fig. 15a. The intersection of the closing lines of the funicular gives the position of the centroid of the five forces engaged. By producing each chord or intercept backward until it intersects the initial line, a series of fresh points are obtained which denote the centers of gravity of the combinations of 1 and 2; 1, 2, and 3, and so on. Verticals through these are next drawn up on the profile so as to intersect the several bases of the corresponding combinations, thus 1, 2, and 3 will intersect the base of lamina 3; and 1, 2, 3, and 4 will intersect that of lamina 4; and so on. These intersections are so many points on the line of pressure (R.E.). The next step is to draw the horizontal forces, i.e., their combinations on to the profile. The process of finding the centers of gravity of these areas is rendered easy by the fact that the combinations are all triangles, not trapezoids, consequently the center of gravity of each is at $\frac{1}{3}$ its height from the base. Thus the center of gravity of the combination $1'+2'+3'$ is at $\frac{1}{3}$ the height measured from the base of 3′ to the apex, in the same way for any other combination, that of 1′, 2′, 3′, 4′, 5′, being at $\frac{1}{3}$ the total height of the profile. The back being vertical, the direction of all the combined forces will be horizontal, and the lines are drawn through, as shown in the figure, to intersect the corresponding combinations of vertical forces. Thus 1′ intersects 1, 1′ 2′ intersects 1 2, and so on. From these several intersections the resultant lines R_1, R_2, to R_5 are now drawn down to the base of the combination to which they belong, these last intersections giving the incidence of R_1, R_2, etc., and are so many points on the line of pressures (R.F.). The process is simple

and takes as long to describe as to perform, and it has this advantage, that each combination of forces is independent of the rest, and consequently errors are not perpetuated. This system can also be used where the back of the profile has one or several inclinations to the vertical, explanation of which will be given later.

28. Actual Pressures in Figures. In the whole process above described, it is noticeable that not a single figure or arithmetical calculation is required. If the actual maximum unit stress due to R or to W is required to be known, the following is the procedure. In Fig. 15a, N scales 174, to reduce this to tons it has to be multiplied by all the eliminated factors, which are $\frac{H}{5}=20$ and $w\rho=\frac{9\times1}{4\times32}$, that is, $N=\frac{174\times20\times9}{4\times32}=244$ tons.

Assuming the incidence of R exactly at the third division, the value of q is $\frac{b}{6}$ and that of m is 2; P also scales 112, its value is therefore $\frac{112\times20\times9}{4\times32}=157$ tons. Applying formula (10_2), $c=N+\frac{\sqrt{N^2+P^2}}{b}=\frac{244+\sqrt{244^2+157^2}}{66.7}=\frac{534}{66.7}=8$ tons per sq. ft., roughly. As 8 tons is obviously well below the limiting stress, for which a value of 16 tons would be more appropriate, this estimation is practically unnecessary but is given here as an example.

29. Analytical Method. The analytical method of calculation will now be worked out for the base of the profile only. First the position of W, the resultant vertical forces (R.E.) relative to the heel of the base will be calculated and next that of R. The back of the profile being in one line and vertical the whole area can be conveniently divided into two right-angled triangles, if the thickening of the curvature at the neck be ignored. As the fore slope has an inclination of $1:\sqrt{\rho}$ the vertical side of the upper triangle (1) is $k\sqrt{\rho}$ in length; its area will then be $\frac{k^2\sqrt{\rho}}{2}=\frac{100\times1.5}{2}=75$ sq. feet. The distance of its c. g. from the heel of the base, which in this case corresponds with the axis of the dam, is $\frac{20}{3}$ feet $=6\frac{2}{3}$ feet.

The moment will then be $\frac{75\times20}{3}=500$. With regard to the lower triangle, its area is $\frac{H^2}{2\sqrt{\rho}}=5000\times\frac{2}{3}=3333.3$ sq. feet. The length of its lever arm is one-third of its base, or 22.2 feet. The moment about the axis will then be $3333.3\times22.2=74{,}000$. The moment of the whole is equal to the sum of the moments of the parts. The area of the whole is $75+3333=3408$. Let x be the required distance of the incidence of W from the heel, then

$$x\times3408=74{,}500$$

$$\therefore \qquad x=\frac{74{,}500}{3408}=21.9 \text{ feet}$$

The inner edge of the middle third is $\frac{b}{3}$ or 22.2 feet distant from the heel; the exact incidence of W is, therefore, .3 foot outside the middle third, a practically negligible amount. With regard to the position of R the distance (f) between the incidence of R and that of W is $\frac{PH}{3W}$; in this P the water-pressure area $=\frac{100\times44.4}{2}=2220$. $\therefore f=\frac{2220\times100}{3\times3408}=21.7$ feet. The total distance of R from the heel will then be $21.7+21.9=43.6$ feet; the outer edge of the middle third is 44.4 feet distant from the heel, consequently the incidence of R is $44.4-43.6=.8$ foot within the middle third, then $q=\frac{b}{6}-.8=\frac{66.7}{6}-.8=10.3$ ft., and $m=\left(1+\frac{6q}{b}\right)=1+.93=1.93$. At this stage it will be convenient to convert the areas into tons by multiplying them by ρw, or $\frac{2.25}{32}$. Then N and W become 239.6, and P becomes 156.3 tons. Formula (**10**) will also be used on account of the high figures; then

$$c=\frac{s}{2}+\sqrt{\frac{s^2}{4}+s_s{}^2}$$

Here $s=\frac{mW}{b}=\frac{239.6\times1.93}{66.7}=6.93$ tons, and $s_s=\frac{P}{b}=\frac{156.3}{66.7}=2.34$ tons, therefore, $c=\frac{6.93}{2}+\sqrt{\frac{6.93^2}{4}+(2.34)^2}=3.46+\sqrt{17.48}=7.64$ tons.

For s_2, or the compression at the heel, $m = 1 - \frac{6q}{b} = .07$. $s_2 = \frac{239.6 \times 0.07}{66.7}$ $= .251$ ton. The area of base pressure is accordingly drawn on Fig. 15.

If W (R.E.) be considered, $q = \frac{b}{6} + .3 = 11.42$, and $m = 1 + \frac{6 \times 11.42}{66.7} = 2.03$; therefore, $s = \frac{mW}{b} = \frac{239.6 \times 2.03}{66.7} = 7.30$ tons. The base pressure is therefore greater with (R.E.) than with (R.F.);

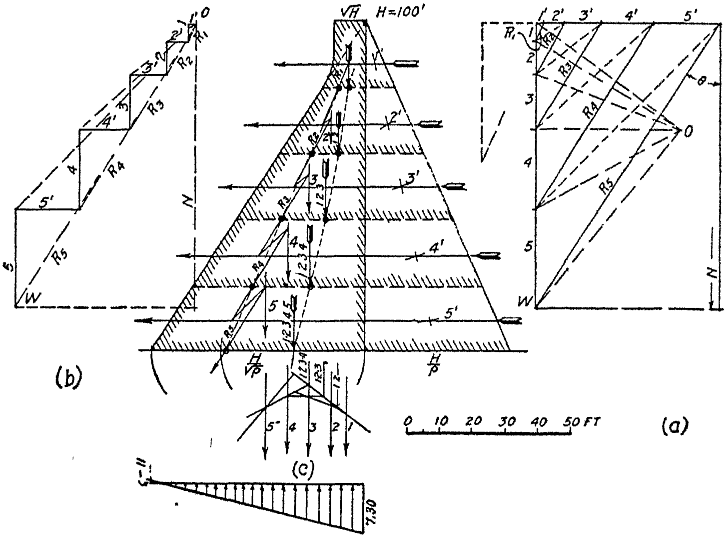


Fig. 16 Diagram Showing Haessler's Method for Locating Lines of Pressure on a Dam

there is also a slight tension at the toe of .11 ton, a negligible quantity. This pressure area is shown on Fig. 16.

30. Haessler's Method. A second method of drawing the line of pressures which is termed "Haessler's" is exhibited in Fig. 16, the same profile being used as in the last example. In this system, which is very suitable for a curved back, or one composed of several inclined surfaces, the forces are not treated as independent entities as before, but the process of combination is continuous from the beginning. They can readily be followed on the force polygon, Fig. 16a and are 1′ with 1 producing R_1; R_1 with 2′, i.e., 1′, 1, 2′, the last resultant being the dotted reverse line. This last is then combined with 2 producing R_2, and so on.

The reciprocals on the profile are drawn as follows: First the c.g.'s of all the laminas 1, 2, 3, etc., 1′, 2′, 3′, etc., are obtained by graphical process. Next the water-pressure lines, which in this case are horizontal, are drawn through the profile. Force line (1′) intersects the vertical (1), whence R_1 is drawn parallel to its reciprocal in Fig. 16a through the base of lamina (1), until it reaches the horizontal force line (2′). Its intersection with the base of (1) is a point in the line of pressure (R.F.). Again from the intersection of R_1 with (2′), a line is drawn backward parallel to its dotted reciprocal line in Fig. 16a until it meets with the second vertical force (2). From this point R_2 is then drawn downward to its intersection with the horizontal force (3′), its intersection with the base of lamina (2) giving another point on the line of pressures. This process is repeated until the intersection of R_5 with the final base completes the operation for (R.F.). It is evident that R_5 as well as all the other resultants are parallel to the corresponding ones in Fig. 15, the same result being arrived at by different graphical processes.

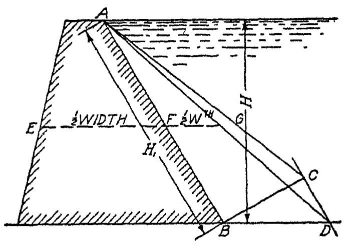


Fig 17 Transformation of Inclined Pressure Area to Equivalent with Horizontal Base

31. Stepped Polygon. Fig. 16b is a representation of the so-called "stepped" polygon, which is also often employed; the form differs, but the principle is identical with that already described. Inspection of the figure will show that all the resultant lines are drawn radiating to one common center or nucleus (O).

The process of finding the incidence of W on the bases of the several lamina is identical with that already described with regard to Fig. 15, viz, the same combination of 1+2, 1+2+3, and so on, are formed in the funicular 16c and then projected on to the profile.

32. Modified Equivalent Pressure Area in Inclined Back Dam. When the back of a dam is inclined, the area of the triangle of water pressure ABC, in Fig. 17, will not equal the product of H, but of H_1 with its half width, which latter is measured parallel to the base, consequently the factor H cannot be eliminated. The triangle itself can, however, be altered in outline so that while containing the same area, it will also have the vertical height H as a factor

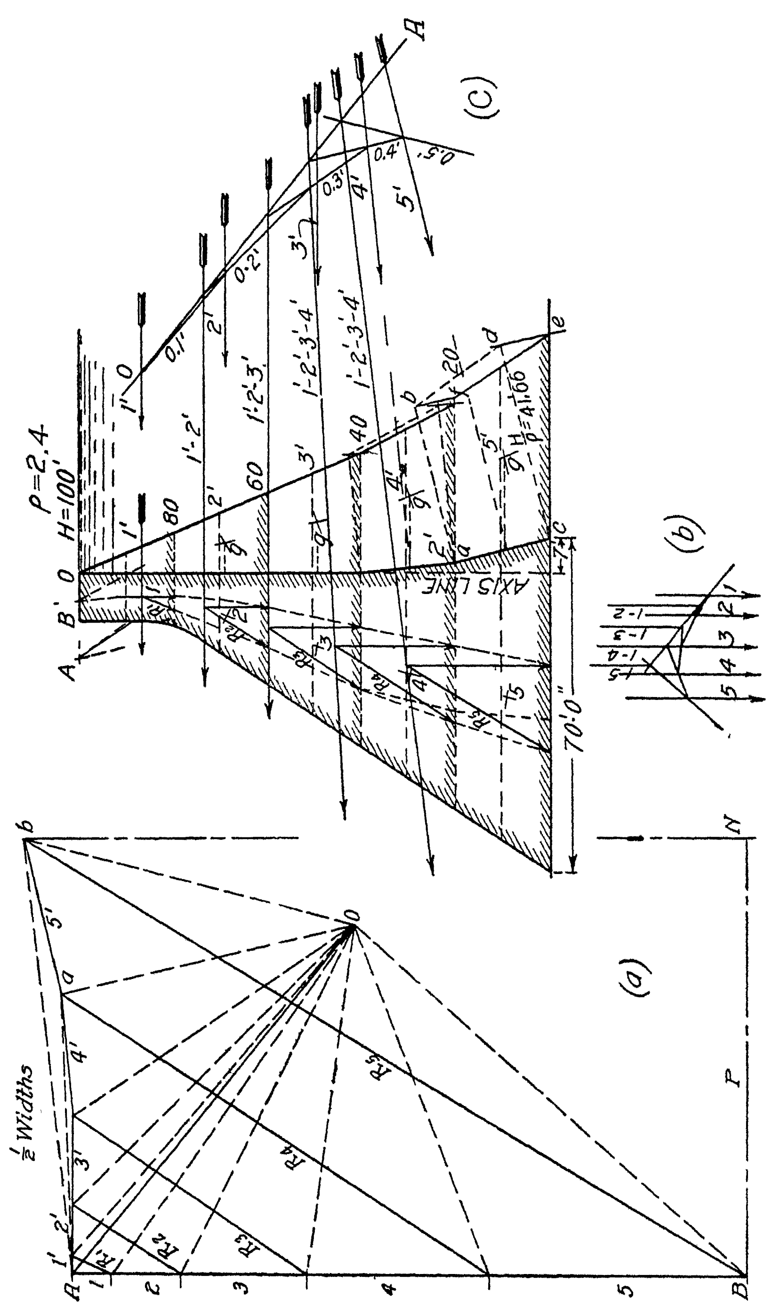


Fig 18 Graphical Method of Fig 15 Applied to Profile with Curved Back

in its area. This is effected by the device illustrated in Fig. 17, and subsequently repeated in other diagrams. In this figure ABC is the triangle of water pressure. By drawing a line CD parallel to the back of the wall AB, a point D is obtained on the continuation of the horizontal base line of the dam. A and D are then joined. The triangle ABD thus formed is equal to ABC, being on the same base AB and between the same parallels. The area of ABD is equal to $\frac{BD}{2} \times H$, and that of the wall to half width $EF \times H$. Consequently we see that the half width $\frac{BD}{2}$ of the triangle ABD can properly represent the area of the water pressure, and the half width EF that of the wall. The vertical height H may, therefore, be eliminated. What applies to the whole triangle would also apply to any trapezoidal parts of it. The direction of the resultant line of water pressure will still be as before, normal to the surface of the wall, i.e., parallel to the base BC, and its incidence on the back will be at the intersection of a line drawn through the c.g. of the area in question, parallel to the base. This point will naturally be the same with regard to the inclined or to the horizontally based area.

33. Curved Back Profiles. In order to illustrate the graphical procedure of drawing the line of pressure on a profile having a curved back, Figs. 18 and 19 are put forward as illustrations merely—not as models of correct design. In these profiles the lower two laminas of water pressure, $4'$ and $5'$, have inclined bases. Both are converted to equivalent areas with horizontal bases by the device explained in the last section. Take the lowest lamina *acdb;* in order to convert it into an equivalent trapezoid with a horizontal base, *de* is drawn parallel to *ac;* the point *e* is joined with A, the apex of the completed triangle, of which the trapezoid is a portion. When *af* is drawn horizontally, the area *acef* will then be the required converted figure, the horizontally measured half width of which multiplied by $\frac{h}{5}$ will equal the area of the original trapezoid *acdb;* $\frac{H}{5}$ can then be eliminated as a common factor and the weights of all the laminas represented in the load line in

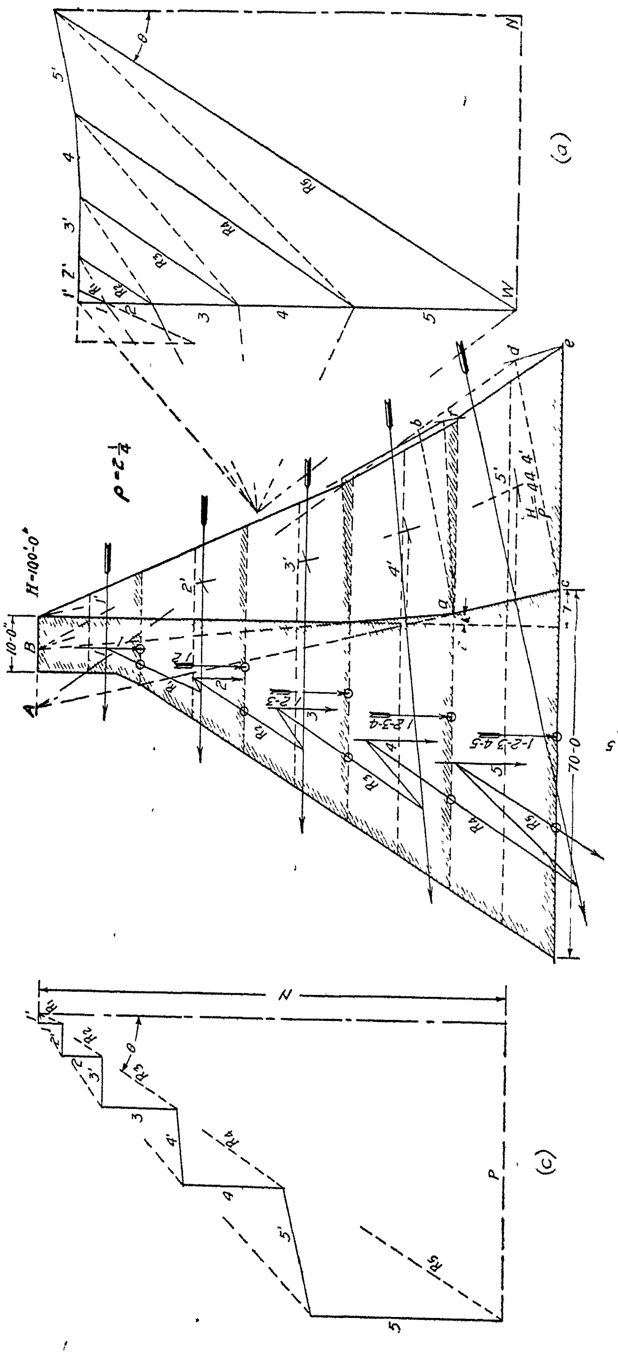


Fig. 19 Haessler's Method of Fig 16 Applied to Curved Back

Fig. 18a, by the half widths of the several areas. The lamina 4′ is treated in a similar manner.

The graphical processes in Figs. 18 and 18a are identical with those in Fig. 15. In the force polygon 18a the water-pressure forces 1′, 2′, 3′, etc., are drawn in directions normal to the adjoining portion of the back of the profile on which they abut, and are made equal in length to the half widths of the laminas in question. The back of the wall is vertical down to the base of lamina 3, consequently the forces, 1′, 2′, and 3′, will be set out on the water-pressure load line in Fig. 18a from the starting point, horizontally in one line. In laminas 4 and 5, however, the back has two inclinations; these forces are set out from the termination of 3′ at their proper directions, i.e., parallel to their inclined bases to points marked *a* and *b*. The direction of the resultants of the combinations, 1′ and 2′, and 1′, 2′, 3′, will clearly be horizontal. If *Aa* and *Ab* be joined, then the directions of the combination 1′ 2′ 3′ 4′ will be parallel to the resultant line *Aa* and that of 1′ 2′ 3′ 4′ 5′ will be parallel to *Ab*. Thus the inclination of the resultant of any combination of inclined forces placed on end, as in the water-pressure load line, will always be parallel with a line connecting the terminal of the last of the forces in the combination with the origin of the load line.

34. Treatment for Broken Line Profiles. The method of ascertaining the relative position and directions of the resultants of water pressure areas when the back of the wall has several inclinations to the vertical is explained as follows: This system involves the construction of two additional figures, viz, a force and ray polygon built on the water-pressure load line and its reciprocal funicular polygon on one side of the profile. These are shown constructed, the first on Fig. 18a, the nucleus *O* of the vertical force and ray polygon being utilized by drawing rays to the terminations of 1′, 2′, 3′, 4′, and 5′. In order to construct the reciprocal funicular polygon, Fig. 18c, the first step is to find the c.g.'s of all the trapezoidal laminas which make up the water-pressure area, viz, 1′ to 5′. This being done, lines are drawn parallel to the bases of the laminas (in this case horizontal lines), to intersect the back of the wall. From the points thus obtained the force lines 1′, 2′, 3′, etc., are drawn at right angles to the portions of the back of the wall on which they abut. On these force lines, which are not all parallel,

the chord polygon (18c) is constructed as follows: First the initial line AO is drawn anywhere parallel to its reciprocal AO, in Fig. 18a. From the intersections of this line with the force line $1'$ the chord marked $O1'$ is drawn parallel to $O1'$ in Fig. 18a and intersecting force line $2'$. Again from this point the chord $O2'$ is drawn intersecting force $3'$ whence the chord $O3'$ is continued to force $4'$, and $O4'$ up to the force line $5'$, each parallel to its reciprocal in Fig. 18a. The closing line is $O5$. The intersection of the initial and the closing lines of the funicular polygon gives the position of the final resultant line $1'\ 2'\ 3'\ 4'\ 5'$, which is then drawn from this point parallel to its reciprocal Ob in Fig. 18a to its position on the profile. The other resultants are obtained in a similar manner by projecting the several chords backward till they intersect the initial line OA, these intersections being the starting points of the other resultants, viz, $1'$-$4'$, $1'$-$3'$, $1'$-$2'$, and $1'$. These resultant lines are drawn parallel to their reciprocals in 18a, viz, $1'$-$4'$ is parallel to Aa, while the remainder are horizontal in direction, the same as their reciprocals.

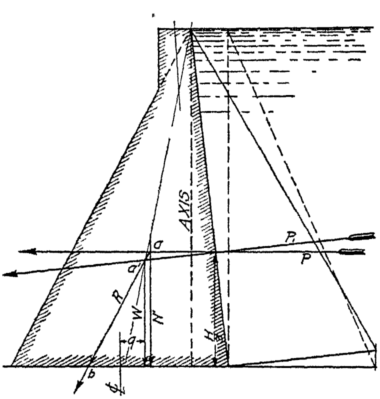


Fig 20. Diagram Showing Third Method of Determining Water Pressure Areas

This procedure is identical with that pursued in forming the funicular 18b, only in this case the forces are not all parallel.

35. Example of Haessler's Method. In Fig. 19 the profile used is similar to Fig. 18, except in the value of ρ, which is $2\frac{1}{4}$, not 2.4 as previously. The graphical system employed is Haessler's, each lamina as already described with reference to Fig. 16 being independently dealt with, the combination with the others taking place on the profile itself. In this case the changes of batter coincide with the divisions of the laminas, consequently the directions of the inclined forces are normal to the position of the back on which their areas abut. This involves finding the c. g.'s of each of the water-pressure trapezoids, which is not necessary in the first system, unless the funicular polygon of inclined forces has to be formed. In spite

of this, in most cases Haessler's method will be found the handiest to employ, particularly in tentative work.

36. Example of Analytical Treatment. In addition to the two systems already described, there is yet another corresponding to the analytical, an illustration of which is given in Fig. 20. In this the vertical and horizontal components of R, the resultants (R.F.), viz, N and P are found. In this method the vertical component of the inclined water pressure P_1 is added to the vertical weight of the dam itself, and when areas are used to represent weights the area of this water overlying the back slope will have to be reduced to a masonry base by division by the specific gravity of the masonry.

37. Relations of R. N. and W. The diagram in Fig. 20 is a further illustration showing the relative positions of R, P, P_1, N, and W. The line R starts from a, the intersection of the horizontal force P with N, the resultant of all the vertical forces, for the reason that it is the resultant of the combination of these two forces; but R is also the resultant of P_1 and W, consequently it will pass through a', the intersection of these latter forces. The points a and a_1 are consequently in the resultant R and it follows as well that if the position of R is known, that of N and W can be obtained graphically by the intersection of P or P_1 with R. These lines have already been discussed.

UNUSUALLY HIGH DAMS

38. "High" Dams. An example will now be given, Fig. 21, of the design of a high dam, i.e., one whose height exceeds the limit before stated. As usual the elementary triangular profile forms the guide in the design of the upper portion. We have seen in section 24 that the limiting depth with $\rho = 2.4$ and $c = 16$ tons = 195 feet, whence for 18 tons' limit the depth will be 219 feet. In Fig. 21 the tentative profile is taken down to a depth of 180 feet. The crest is made 15 feet wide and the back is battered 1 in 30; the base width is made $180 \times .645 = 116$ feet. The heel projects 6 feet outside the axis line. The graphical procedure requires no special explanation. It follows the analytical in dealing with the water pressure as a horizontal force, the weight of the water overlying the back being added to that of the solid dam. For purposes of

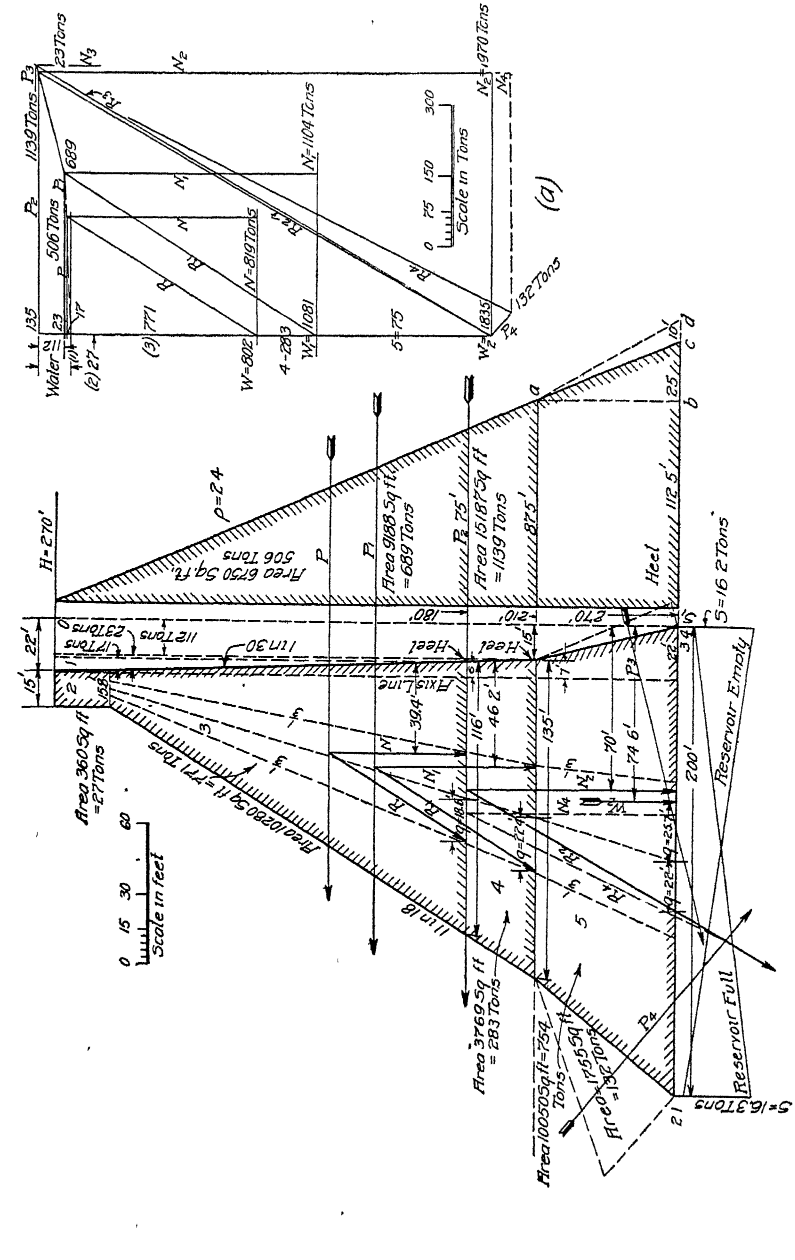


Fig 21 Analytical Diagram for "High" Dam

calculation the load is divided into three parts (1) the water on the sloping back, the area of which is 540 sq. ft. This has to be reduced by dividing it by ρ and so becomes 225 sq. ft. As tons, not areas, will be used, this procedure is not necessary, but is adopted for the sake of uniformity in treatment to avoid errors. The c.g. of (1) is clearly 2 feet distant from the heel of the base of (3), about which point moments will be taken. That of the crest (2) is 13.3 feet and that of the main body (3) obtained by using formula (**7**) comes to 41.2 feet. The statement of moments is then as follows:

No.	Area	Tons	Lever Arm	Moment
1	225	17	2	34
2	360	27	13.3	359
3	10280	771	41.2	31765
Total	10866	815		32158

Then the distance of N from the heel will be $\frac{32158}{815}=39.4$ ft.; it thus falls $39.4-38.7=.7$ ft. within the middle third. The distance (f) between N and R is $\frac{PH}{3N}$. Now $P=$the area of the right angle triangle whose base is $\frac{H}{\rho}$, or 75 feet, and is 6750 sq. ft. equivalent to $\frac{6750\times 3}{40}=506$ tons. The expression then becomes $f=\frac{506\times 180}{3\times 815}$ $=37.2$ feet. The incidence of R will then be $37.2+39.4=76.6$ feet distant from the heel of the base. The $\frac{2}{3}$ point being 77.3 from the heel, R falls .7 ft. within. Thus far the tentative profile has proved fairly satisfactory, although a slight reduction in the base width is possible. The position of W, or the resultant weight of the portions 2 and 3 of the dam is obtained from the moment table already given, and is the sum of the moments of (2) and (3) divided by (2) $+$(3) or $\frac{32124}{798}=40.2$ ft. This falls $40.2-38.7=1.5$ ft. within the

middle third. The value of q (R.F.) is $\frac{b}{6}-.7=19.33-.7=18.6$ feet, and $m=\left(1+\frac{6q}{b}\right)=1+\frac{111.6}{116}=1.96.$

Then by formula (10_1), N being 815 and P, 506

$$c=\frac{1.96\times815+\sqrt{(1.96\times815)^2+4\ (506)^2}}{232}$$

$$=\frac{1597+\sqrt{2551367+1024144}}{232}$$

$$=15.03 \text{ tons per square foot}$$

Extension of Profile. This value being well below the limit of 18 tons and both resultants (R.F.) and (R.E.) standing within the middle third it is deemed that the same profile can be carried down another 30 ft. in depth without widening. The base length will now be a trifle over 135 feet. The area of the new portion (4) is 3769 sq. ft. =283 tons. The distance of its c.g. from the heel by formula (7) is found to be 63.4 feet. The position of W will be obtained as follows, the center of moments being one foot farther to right than in last paragraph.

No	Tons	Lever Arm	Moment
2	27	14.3	386
3	771	42.2	32536
4	283	63 4	17942
Total	1081		50864

$\therefore \qquad x=\frac{50864}{1081}=47.0$ feet from heel of new base to W

As $\frac{b}{3}$ is $\frac{135}{3}=45$ ft., the incidence of W_1 is 2.0 ft. within the base, which is satisfactory.

To find N_1, the moment of the water on whole back can be added to that of W first obtained. The offset from the axis being now 7 ft., the area will be $\frac{210\times7}{2}=735$ equal to an area of masonry of $\frac{735}{2.4}=306$ sq. ft. equivalent to 23 tons, nearly. The lever arm

being $\frac{7}{3}$, or 2.3 feet the moment about the heel will be $23\times 2.3=52.9$, say 53.0 ft.-tons. This amount added to the moment of W_1 will represent that of N_1 and will be $50{,}864+53=50{,}917$. The value of N_1 is that of W_1+ the water on back or, $1081+23=1104$ tons. The distance of N_1 from the heel is then $\frac{50917}{1104}=46.2$ feet. To obtain that of R_1 the value of $f=\frac{689\times 210}{3\times 1104}=43.7$ feet; this added to $46.2=89.9$ ft., the incidence of R_1 is therefore $\frac{135\times 2}{3}-89.9=90-89.9=.1$ ft., within the middle third boundary. Then $q=\frac{b}{6}-.1=22.5-.1=22.4$ ft., and $m=1+\frac{6q}{b}=1+\frac{134.4}{135}=2.00$, nearly. To find c, formula (**10**) will be used, the quantities being less than in formula (**10**$_1$). Here $s=\frac{mN}{b}=\frac{2.00\times 1104}{135}=16.4$ tons and $s_s=\frac{P}{b}=\frac{689}{135}=5.1$ tons. Then

$$c=\frac{16.4}{2}+\sqrt{\frac{(16.4)^2}{4}+(5.1)^2}=8.2+9.7=17.9 \text{ tons}$$

The limit of 18 tons, being now reached, this profile will have to be departed from.

39. Pentagonal Profile to Be Widened. The method now to be adopted is purely tentative and graphic construction will be found a great aid to its solution. A lamina of a depth of 60 ft., will be added to the profile. It is evident that its base width must be greater than that which would be formed by the profile being continued down straight to this level. The back batter naturally will be greater than the fore. From examination of other profiles it appears that the rear batter varies roughly from about 1 in 5 to 1 in 8 while the fore batter is about 1 to 1. As a first trial an 8 ft. extra offset at the back was assumed with a base of 200 feet; this would give the required front projection. Graphical trial lines showed that N would fall without the middle third, and W as well; the stress also just exceeded 18 tons (R.E.). A second trial was now made in which the back batter was increased and the base shortened to 180 feet. In this case c exceeded the 18 ton limit.

Still further widening was evidently required at the heel in order to increase the weight of the overlying water, while it was clear that the base width would not bear reduction. The rear offset was then increased to 15 feet and the base width to 200 feet. The stresses now worked out about right and the resultants both fell within the middle third. By using formula (6) the distance of the c.g. of the trapezoid of water pressure, which weighs 112 tons, was found to be 7.2 feet from the heel of the base, and by formula (7) that of the lowest lamina (5) from the same point is found to be 91.8 feet; the weight of this portion is 754 tons. These two new vertical forces can now be combined with N_1 whose area and position are known and thus that of N_2 can be ascertained. N_1 is 46.2 ft. distant from the heel of the upper profile; its lever arm will, therefore, be $46.2+15=61.2$ feet. The combined moment about the heel will then be

	Weight	Lever Arm	Moment
Water	112	7.2	806
N_1	1104	61.2	67565
(5)	754	91.8	69217
Total	1970		137588

The incidence of N_2 is then $\frac{137588}{1970}=70$ feet from the heel; as the middle third boundary is $\frac{200}{3}=66.6$ feet distant from the same point, N_2 falls 3.4 feet within. The distance between N_2 and R_2 (viz, f) $=\frac{PH_2}{3N_2}$. Now P_2, or the horizontal water pressure, has a reduced area of 15187 feet, equivalent to 1139 tons, consequently $f=\frac{1139\times 270}{3\times 1970}$ $=52.0$ feet. This fixes the incidence of R_2 at $70+52.0=122.0$ feet distant from the heel; the $\frac{2}{3}$ point is 133.3 feet distant, consequently R_2 falls well within the middle third and as $q=122.0-\frac{200}{2}=22.0$ feet, $m=1+\frac{6\times 22.0}{200}=1.66$, and $s=\frac{mN}{b}=\frac{1.66\times 1970}{200}=16.3$ tons.

Now $$s_s=\frac{P_2}{b}=\frac{1139}{200}=5.7 \text{ tons}$$

Whence by formula (**10**),

$$c=\frac{16.3}{2}+\sqrt{\frac{265}{4}+32.5}=8.15+9.9=18.05 \text{ tons}$$

which is the exact limit stress.

The value of s_2 (the pressure at the heel) is obtained by the same formula, using the minus sign, viz, $m=1-\frac{130.8}{200}=.34$, therefore, $s_2=\frac{mN_2}{b}=\frac{.34\times 1970}{200}=3.4$ tons, nearly. These vertical reactions are set out below the profile. With regard to W_2 it is composed of $W_1+(5)$. The table of moments is as follows:

	Weight	Lever Arm	Moment
W_1	1081	62.0	67022
(5)	754	91 8	69217
W_2	1835		136239

The distance of W_2 from the heel is $\frac{136239}{1835}=74.3$. W_2, therefore, falls 7.6 feet within the middle third; $q=\frac{200}{2}-74.3=25.7$ feet, and $m=1+\frac{6\times 25.7}{200}=1.77$, and $m=1-.77=.23$; therefore, $s=\frac{mW_2}{b}=\frac{1.77\times 1835}{200}=16.2$ tons, and $s_2=.23\times\frac{1835}{200}=2.1$ tons. These pressures are shown below the profile.

The value of θ in all three cases is less than 35° which is also one of the stipulations.

In continuing the profile below the 270-foot depth the probability is that for a further depth of 50 or 60 feet the same fore and rear batter would answer; if not, the adjustment is not a difficult matter to manipulate. As previously stated, the incidence of N should be fixed a little within the middle third when that of W and R will generally be satisfactory.

In the force diagram the water part of N is kept on the top of the load line W. This enables the lengths of the N series to be clearly shown. The effect is the same as if inclined water pressure lines were drawn, as has already been exhibited in several cases.

40. Silt against Base of Dam. In Fig. 21, suppose that the water below the 210-foot depth was so mixed with silt as to have a specific gravity of 1.4 instead of unity. The effect of this can be shown graphically without alteration of the existing work. In the trapezoid lying between 210 and 270 the rectangle on ab represents the pressure above 210 and the remaining triangle that of the lower 60 feet of water. The base of the latter, bc is, therefore, $\frac{H}{\rho}=\frac{60}{2.4}=$ 25 feet. Now the weight of the water is increased in the proportion of 1.4 : 1, consequently the proper base width will be $\frac{H' \times 1.4}{2.4}=$ $\frac{60 \times 1.4}{2.4}=35$ feet. The triangle acd then represents the additional pressure area due to silt. The normal pressure on the back of the dam due to the presence of silt is shown graphically by the triangle attached, whose base $=cd=10$ feet; its area is 310 square feet, equivalent to 23 tons. This inclined force is combined with R_2 at the top right-hand corner of Fig. 21a and the resultant is R_3; on the profile the reciprocal inclined force is run out to meet R_2 and from this intersection R_3 can be drawn up toward P_2. This latter intersection gives the altered position of N_2, which is too slight to be noticeable on this scale. The value of c and the inclination of R are both increased, which is detrimental.

If the mud became consolidated into a water-tight mass the pressure on the dam would be relieved to some extent, as the earth will not exert liquid pressure against the back. Liquid mud pressure at the bottom of a reservoir can consequently be generally neglected in design.

41. Filling against Toe of Dam. Now let the other side of the dam be considered. Supposing a mass of porous material having an immersed s. g. of 1.8 is deposited on the toe, as is often actually the case. Then a pressure triangle of which the base equals $H \times \frac{1.8}{2.4}=$ 45 feet is drawn; its area will be 1755 and weight 132 tons; the

resultant P_4 acting through its c. g. is run out to intersect R_2. At the same time from the lower extremity of R_2, in the force diagram, a reciprocal pressure line P_4 is drawn in the same direction equal in length 132 tons and its extremity is joined with that of P_2; the resulting line R_4 is then projected on the profile from the previous intersection until it cuts the force line P_2; this gives a new resultant R_4 and a new position for N, viz, N_4, which is drawn on the profile; W also will be similarly affected. The load on the toe of the dam increases its stability as the value of θ is lessened, the position of W is also improved, but that of R_4, which is nearer to the toe than R_3, is not. To adjust matters, the c. g. of (5) requires moving to the right which

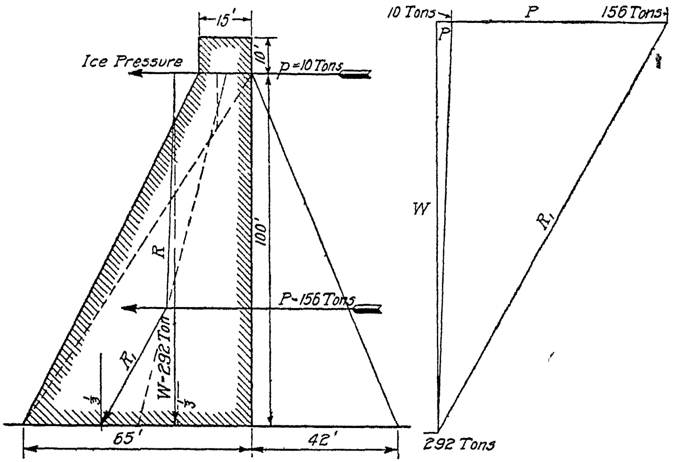


Fig 22 Diagram Showing Effect of Ice Pressure

is affected by shifting the base line thus increasing the back and decreasing the front batter, retaining the base length the same as before.

42. Ice Pressure. Ice pressure against the back of a dam has sometimes to be allowed for in the design of the profile; as a rule, however, most reservoirs are not full in winter so that the expansive pressure is exerted not at the summit but at some distance lower down where the effect is negligible. In addition to this when the sides of a reservoir are sloping, as is generally the case, movement of ice can take place and so the dam is relieved from any pressure. In the estimates for the Quaker Bridge dam it is stated that an ice pressure of over 20 tons per square foot was provided for. No

definite rules seem to be available as to what allowance is suitable. Many authorities neglect it altogether.

The effect of a pressure of ten tons per foot run on a hundred-foot dam acting at the water level is illustrated in Fig. 22. For this purpose a trapezoidal section has been adopted below the summit level. The crest is made 15 feet wide and 10 feet high. This solid section is only just sufficient, as will appear from the incidence of R' on the base. The area of this profile is 4150 sq. ft., while one of the ordinary pentagonal sections as dotted on the drawing would

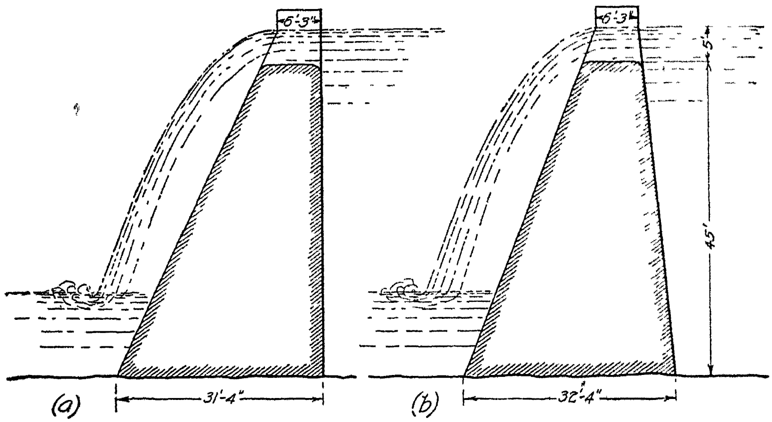


Fig 23. Two Profiles for Partial Overfall Dams

contain but 3325 sq. ft. The increase due to the ice pressure is therefore 825 sq. ft. or about 25 per cent. The graphical procedure hardly needs explanation. The ice pressure p is first combined with W the weight of the dam and their resultant R cuts P at a point from which the final R_1 is run down to the base parallel to its reciprocal in the force diagram. It falls just within the middle third of the base. An actual example is given in section 56.

43. Partial Overfall Dams. It not infrequently happens that the crest of a dam is lowered for a certain length, this portion acting as a waste weir, the crest of the balance of the dam being raised above the water level. In such cases a trapezoidal outline is generally preferable for the weir portion and the section can be continued upon the same lines to form the upper part of the dam, or the upper part can be a vertical crest resting on the trapezoidal body. In a

trapezoidal dam, if the ratio of $\frac{k}{b}$ be r, the correct base width is obtained by the following formula:

$$b=\frac{H}{\sqrt{\rho}}\,\frac{1}{\sqrt{1+r-r^2}} \qquad \textbf{(14)}$$

This assumes the crest and summit water level to be the same. In Fig. 23, ρ is taken as 2.4 and r as .2. The base width with a vertical back will then be $\frac{H}{\sqrt{\rho}}\times\frac{1}{\sqrt{1+.2-.04}}=50\times.645\times.935=31.3$ feet, and the crest width k will be $31.3\times.2=6.3$ feet. In the second figure the profile is shown canted forward, which is desirable in weirs, and any loss in stability is generally more than compensated for by the influence of the reverse pressure of the tail water which influence increases with the steepness of the fore slope of the weir. The base width is, however, increased by one foot in the second figure.

As will be seen in the next section, the crest width of a weir should not be less than $\sqrt{H}+\sqrt{d}$; in this case $H=45$ and $d=5$. This would provide a crest width of $6.7+2.2=9$ feet, which it nearly scales.

NOTABLE EXISTING DAMS

44. Cheeseman Lake Dam. Some actual examples of dam sections will now be exhibited and analyzed. Fig. 24 is the section of the Cheeseman Lake dam near Denver, Colorado, which is one of the highest in the world. It is built to a curvature of 400 feet radius across a narrow canyon. It is considered a gravity dam, however, and will be analyzed as such. The section can be divided into three unequal parts 1, 2, and 3, and the lines of pressure (R.F.) and (R.E.) will be drawn through the bases of these three divisions. Of the vertical forces (1) has an area of 756 sq. ft., (2) of 3840, and (3) of 13,356 the total value of W being 17,952 sq. ft., which is marked off on the load line in Fig. 24a. With regard to the water-pressure areas the most convenient method, where half widths are not used, which can only be done with equal divisions, is to estimate the areas of the horizontal pressures only and set them off horizontally, the values of the inclined pressures being obtained by construction. For this purpose the triangle of horizontal water pressure is shown adjacent to, but separate from, the profile. The three values of P

which are equal to $\frac{H^2}{2\rho}$ will be 270, 2631, and 7636, respectively, the total being 10,537 sq. ft. In this computation the value of ρ is assumed to be 2.4. These several lengths are now set out horizontally from the origin O in Fig. 24a, and verticals drawn upward intercept the chords, $1'$, $2'$, $3'$, which latter are drawn from the origin O, parallel to their respective directions, i.e., normal to the adjacent parts of the wall. The rest of the process is similar to that already described, with reference to Figs. 16 and 18, and need not be repeated. In Fig. 24a N scales 19,450, equal to 1457 tons, and

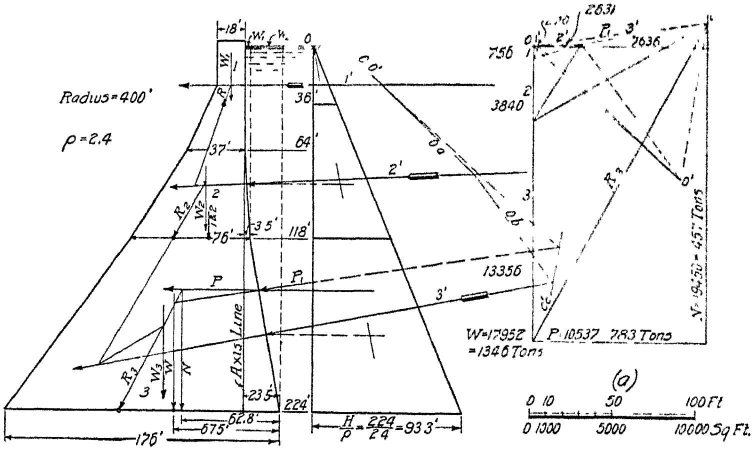


Fig 24 Profile of Cheeseman Lake Dam

on the profile q scales 15 feet, therefore, in formula (9), $m=1+\frac{90}{176}=1.51$. Therefore, $s=\frac{mN}{b}=\frac{1.51\times1457}{176}=12.5$ tons, and $s_s=\frac{P}{b}=\frac{783}{176}=4.45$; then by formula (10)

$$c=\frac{12.5}{2}+\sqrt{\frac{(12.5)^2}{4}+(4.45)^2}=6.25+\sqrt{59}=13.9 \text{ tons, approx.}$$

With regard to W, q scales about 20 ft., m then works out to 1.7, nearly, and $s=\frac{mW}{b}=\frac{1.7\times1346}{176}=13.0$ tons.

As an exercise the inclined final resultant P is drawn on the profile. This line is parallel to Oc in Fig. 24a, its location is worked out by means of the funicular polygon, the construction of which need not be explained after what has gone before.

45. Analytical Check. In order to check this result analytically the procedure will be, first, calculate the position of the c.g. of the trapezoids (2) and (3) relative to the rear corner of their bases by formula (7) and also the positions of the resultants of the vertical components of the water pressure overlying the back with regard to the same points by formula (6). Second, convert the areas into tons by multiplying by $\frac{3}{40}$. The statement of moments about the heel of the base, with the object of finding the position of W is given below.

Moment of (1)	56.7 ×32.5	= 1843
Moment of (2)	288 ×47.9	= 13795
Moment of (3)	1001 ×75	= 75075
Total W =	1346 tons	90713

The distance of W from the heel will then be $\frac{90713}{1346}=67.5$ ft. In order to obtain N, the moments of the water weights will have to be added as below.

Moment of W	1346×67.5	= 90713
Moment of w_1	10×21.6	= 216
Moment of w_2	107×9	= 963
Total N=	1463 tons	91892

and

$$x=\frac{91892}{1463}=62.8 \text{ feet}$$

To find the incidence of R and its distance (q) from the center point, that from the known position of N must be computed from the formula $f=\frac{PH}{3N}=\frac{783\times 224}{3\times 1463}=40$ ft., therefore, $q=(62.8+40.0)-\frac{176}{2}=14.8$ feet. This is close to the value obtained graphically which was taken as 15 feet. The value of N is also seen to be close to that obtained graphically. The value of q with regard to W (R.E.) is as follows, $q=\frac{176}{2}-67.5=20.5$ feet, almost exactly what it scales on

the diagram. In this profile the upper part is light, necessarily made up for in the lower part.

At the upper base line of (2) the incidence of W is exactly at the middle third edge, while R falls within it. At the final base the position of N is 62.8 distant from the heel and the inner third point is $\frac{176}{3} = 58.6$ distant, consequently the incidence of N lies 4.2 feet within the boundary.

If the position of N were made obligatory at the inner edge of the middle third, the value of W would be increased, but R would

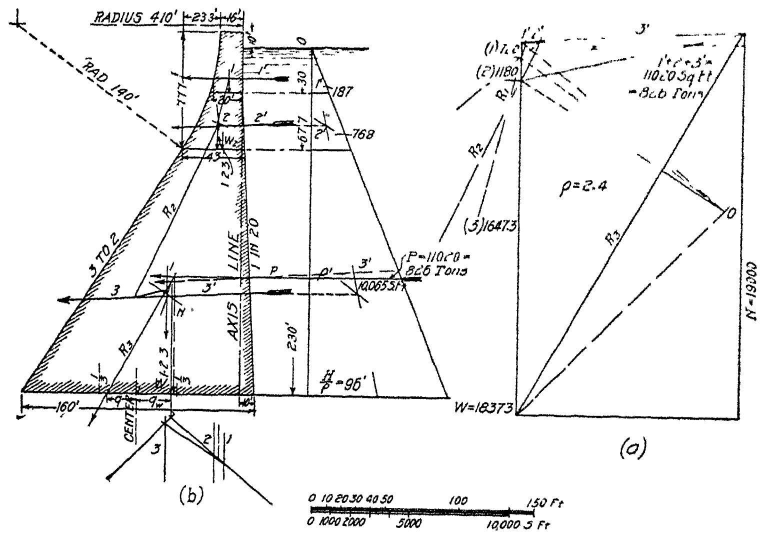


Fig 25. Profile of Roosevelt Dam across Salt River, Arizona

be decreased. There may have been special reasons for limiting the maximum stress (R.E.). On Fig. 24 the position of N is obtained by the intersection of the horizontal resultant P with R prolonged upward. If the stress were calculated on the supposition that the structure was an arched dam, it would amount to $21\frac{1}{4}$ tons by the "long" formula, given in section 78, Part II.

46. Roosevelt Dam. In Fig. 25 is given the profile of the Roosevelt dam, and Fig. 26 is the site plan of that celebrated work. For some years, the Roosevelt dam was the highest gravity dam in existence. It spans a very deep canyon of the Salt River in Arizona

and impounds the enormous quantity of $1\frac{1}{4}$ million acre-feet of water, which will be utilized for irrigation. This work is part of one of the greatest of the several large land reclamation projects undertaken by the U. S. Government for the watering and settling of arid tracts in the dry zone of the western states.

The profile is remarkable for the severe simplicity of its outline. It closely follows the elementary profile right down to its extreme base and forms a powerful advocate for this simple style of design. The graphical procedure is similar to that in the last example. The section is divided into three divisions. As the first two are comparatively small, the triangle of forces in Fig. 25a

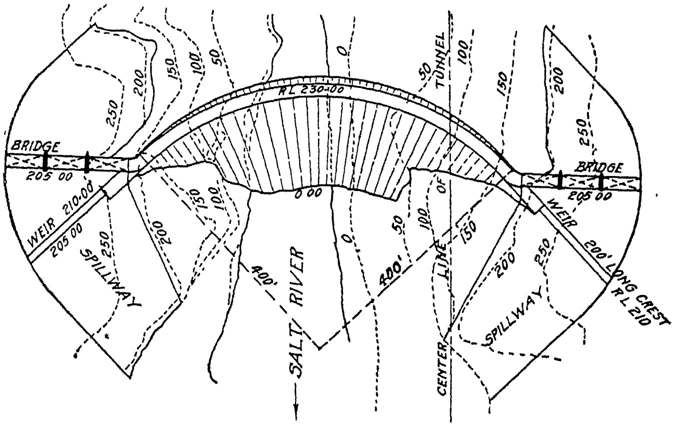


Fig 26 Site Plan for Roosevelt Dam

is first plotted at a large scale in pencil and the inclinations of the resultants thus obtained are transferred to the profile; this accounts for the long projecting lines near the origin of the force diagram which also appear in some previous examples. A neater method for overcoming this difficulty is that adopted in the next figure, when the forces (1) and (2) are first amalgamated into one before being plotted on the force diagram.

In Fig. 25a, N scales roughly 19,000 sq. ft., equivalent to 1425 tons, q also measures approximately 20 ft., then $m=1+\frac{120}{160}=1.75$, and $s=\frac{mN}{b}=\frac{1425\times1.75}{160}=15.5$ tons. $s_s=\frac{P}{b}=\frac{826}{160}=5.1$ tons. By formula (10)

$$c=\frac{15.5}{2}+\sqrt{\frac{(15.5)^2}{4}+(5.1)^2}=7.75+\sqrt{86}=17 \text{ tons roughly}$$

With regard to W, q measures 23 ft. and m works out to 1.86 therefore $s=\frac{mW}{b}=\frac{1.86\times1378}{160}=16$ tons per sq. ft.

This dam is built on a radius of 410 feet, measured from the axis; if measured from the extrados of the curve at the base it will be 420 feet and the arch stress as calculated from the "long" formula used in "Arched Dams" will amount to 23.3 tons.

The site plan given in Fig. 26 forms an instructive example of the arrangement of spillways cut in the solid rock out of the shoulders of the side of the canyon, the material thus obtained being used in the dam. These spillways are each 200 feet wide and are excavated down to five feet below the crest of the dwarf waste weir walls which cross them. This allows of a much greater discharge passing under a given head than would be the case with a simple channel without a drop wall and with bed at the weir crest level. The heading up, or afflux, is by this means diminished and that is a matter affecting the height given to the dam crest.

47. New Croton Dam. The profile of the New Croton dam, constructed in connection with the water supply of New York City is given in Fig. 27. This dam has a straight alignment and is 1168 feet long. Waste flood water is accommodated by an overfull weir 1000 ft. in length, which is situated on one flank forming a continuation of the dam at right angles to its axis. The surplus water falls into the Rocky River bed and is conveyed away by a separate channel. An elevation and plan of this work are given in Figs. 28 and 29.

The system of graphical analysis employed in this case is different from that in the last two examples and is that illustrated in Fig. 18, where independent combinations of vertical and inclined forces are used. The profile is divided into four divisions, the first being a combination of two small upper ones. The further procedure after the long explanations already given does not require any special notice except to point out that the directions of the combined forces $1'$, $1'+2'$, $1'+2'+3'$ etc., in (d) are drawn parallel to their reciprocal lines on Fig. 27a, namely to the chords Oa, Ob, Oc, and Od, respectively. The final resultants are R_4 (R.F.) and W (R.E.). The

value of W is 1380 tons and that of N is 1484 tons, consequently applying formula (**10**), q in the first case scales 26 feet and m works out to 1.82 therefore $s=\frac{mW}{b}=\frac{1.82\times1380}{190}=13.2$ tons $=c$, as with W, s and c are identical.

With regard to N, q scales 7 feet, consequently $m=1+\frac{42}{190}=1.22$, $s=\frac{1.22\times1484}{190}=9.5$ tons only. As $P=10{,}010=750$ tons, $\frac{P}{b}=4$ tons; therefore $c=\frac{9.5}{2}+\sqrt{\frac{(9.5)^2}{4}+(4)^2}=11$ tons, which is very moderate. It is probable that other external pressures exist due

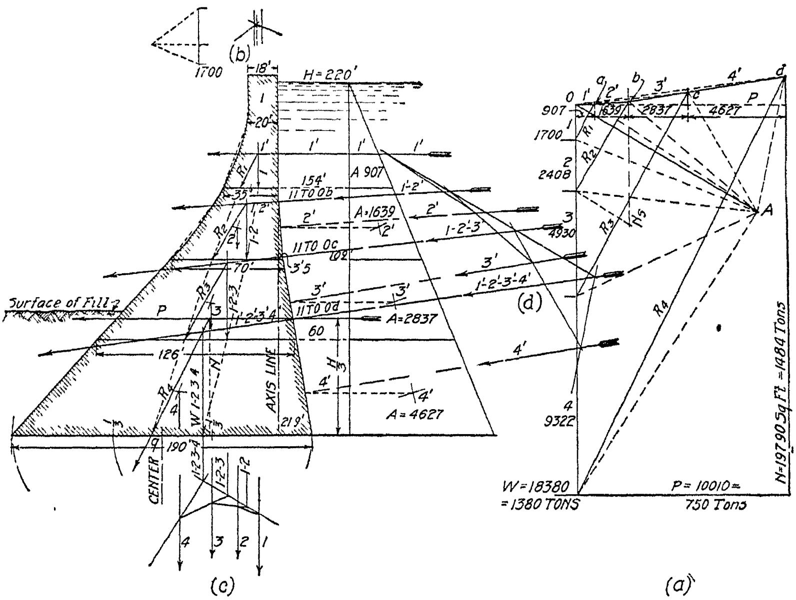


Fig 27. Diagram of Profile of New Croton Dam Showing Influence Lines as in Fig 18

to filling in front and rear, as also ice pressure, which would materially modify the result above shown. This dam, like the Cheeseman, is of the bottleneck profile, it is straight and not curved on plan.

48. Assuan Dam. The section, Fig. 30, is of the Assuan dam in Egypt, which notable work was built across the Nile River above the first cataract. As it stands at present it is not remarkable for

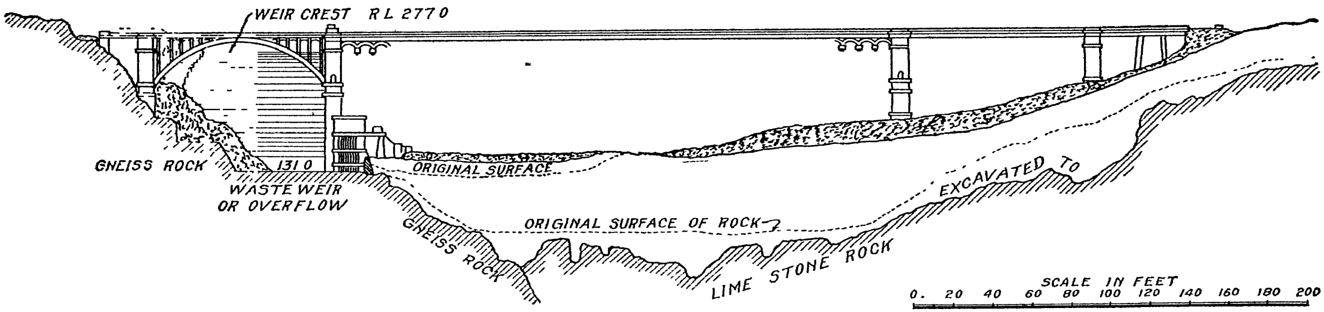


Fig. 28 Elevation of New Croton Dam Constructed in Connection with Water Supply of New York City

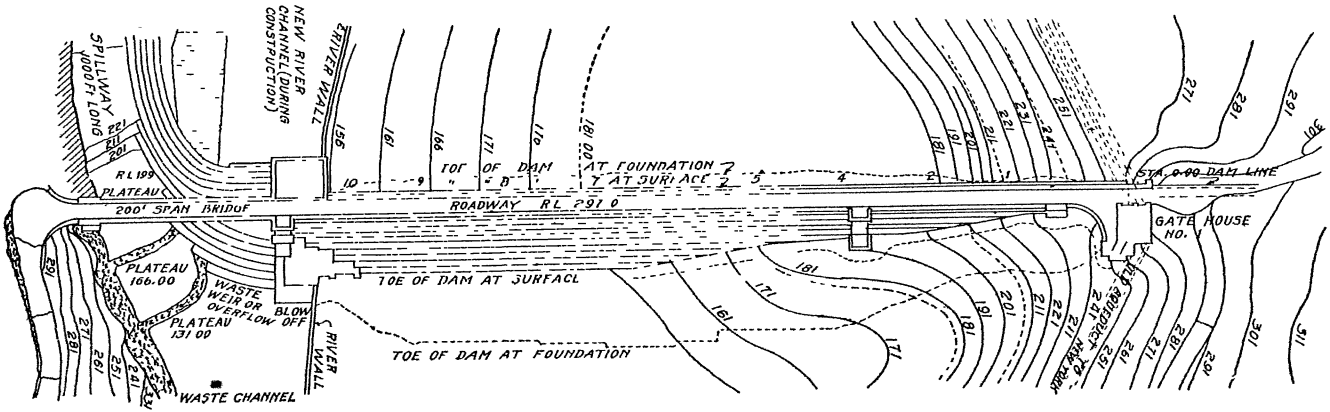


Fig 29. Plan of New Croton Dam for New York City Water Supply

its height, but what it lacks in that respect, as in most eastern works, is made up in length, which latter is 6400 feet. No single irrigation work of modern times has been more useful or far-reaching in beneficial results upon the industrial welfare of the people than this dam. Its original capacity was 863,000 acre-feet and the back water extended for 140 miles up the river. The work is principally remarkable as being the only solid dam which passes the whole discharge of a large river like the Nile, estimated at 500,000 second-feet, through its body, for which purpose it is provided with 140 low and 40 high sluices. These are arranged in groups of ten, each

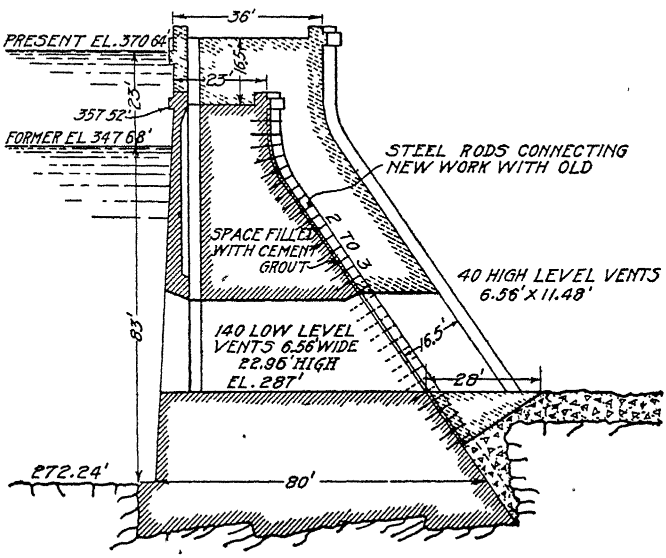


Fig 30 Assuan Dam across the Nile Showing Old and New Profiles

low sluice is 23 feet deep by $6\frac{1}{2}$ feet wide with the dividing piers $16\frac{1}{2}$ feet wide. The diminution of the weight of the dam due to sluices necessitates an excess of width over what would be sufficient for a solid dam; in addition to which the maximum pressure in the piers is limited to the extremely low figure of 5 tons of 2000 pounds. The designers have thus certainly not erred on the side of boldness; the foundation being solid granite, would presumably stand, with perfect safety, pressure of treble that intensity, while the masonry, being also granite, set in cement mortar, is certainly capable of carrying a safe pressure of 15 tons, as many examples prove.

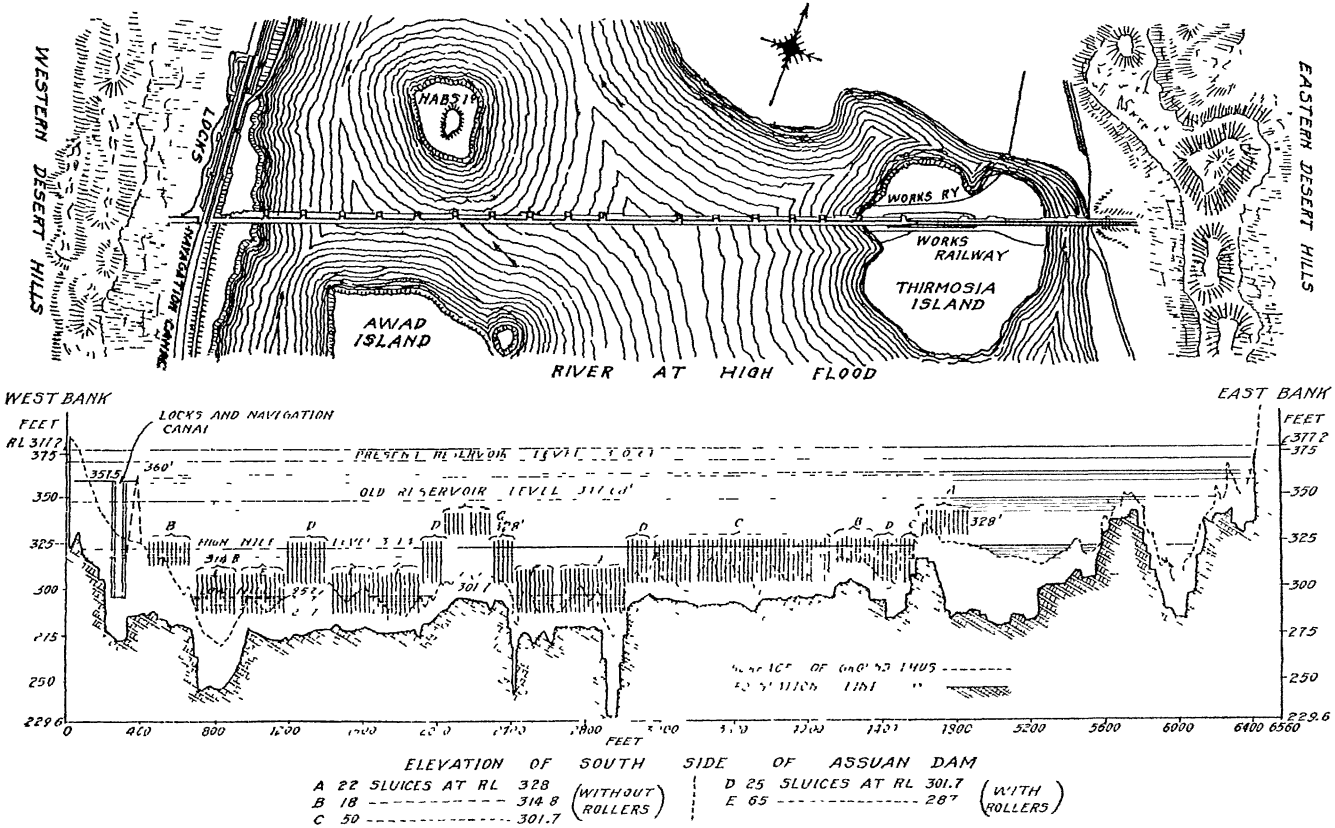


Fig 31 Location Plan and Longitudinal Section of Assuan Dam

This dam has proved such a financial success that it has recently been raised by 23 feet to the height originally projected. The water thus impounded is nearly doubled in quantity, i.e., to over $1\frac{1}{2}$ million acre-feet; exceeding even that of the Salt River reservoir in Arizona. As it was decided not to exceed the low unit pressure previously adopted, the profile has been widened by $16\frac{1}{2}$ feet throughout. A space has been left between the new and the old work which has been subsequently filled in with cement grout under pressure, in addition to which a series of steel rods has been let

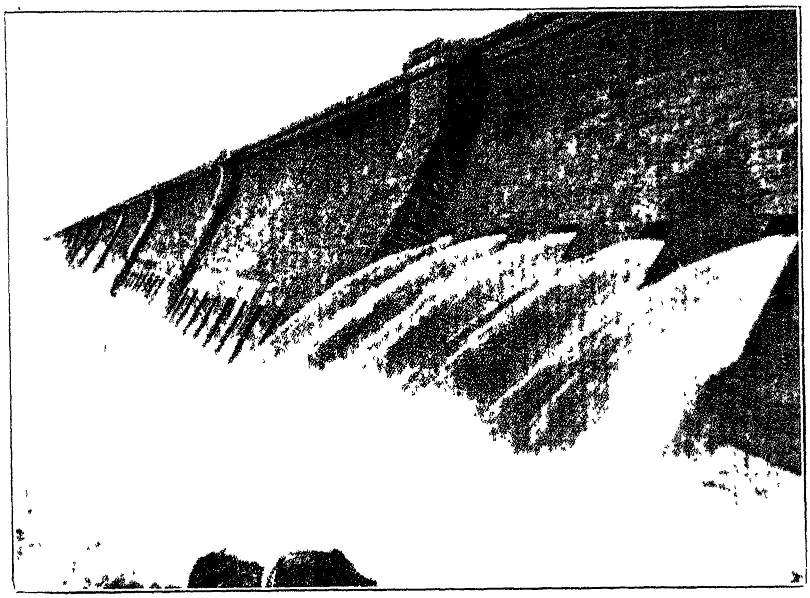


Fig. 32. View of Assuan Dam before Being Heightened with Sluices in Operation

into the old face by boring, and built into the new work. The enlargement is shown in the figure. The sluices are capable of discharging 500,000 second feet; as their combined area is 25,000 square feet this will mean a velocity of 20 feet per second. Owing, however, to the possibility of adjustment of level, by manipulation of the sluice gates, they will never be put to so severe a test.

A location plan and longitudinal section shown in Fig. 31, a view of the sluices in operation, Fig. 32, and a view of the new work in process, Fig. 33, will give a good idea of the construction features.

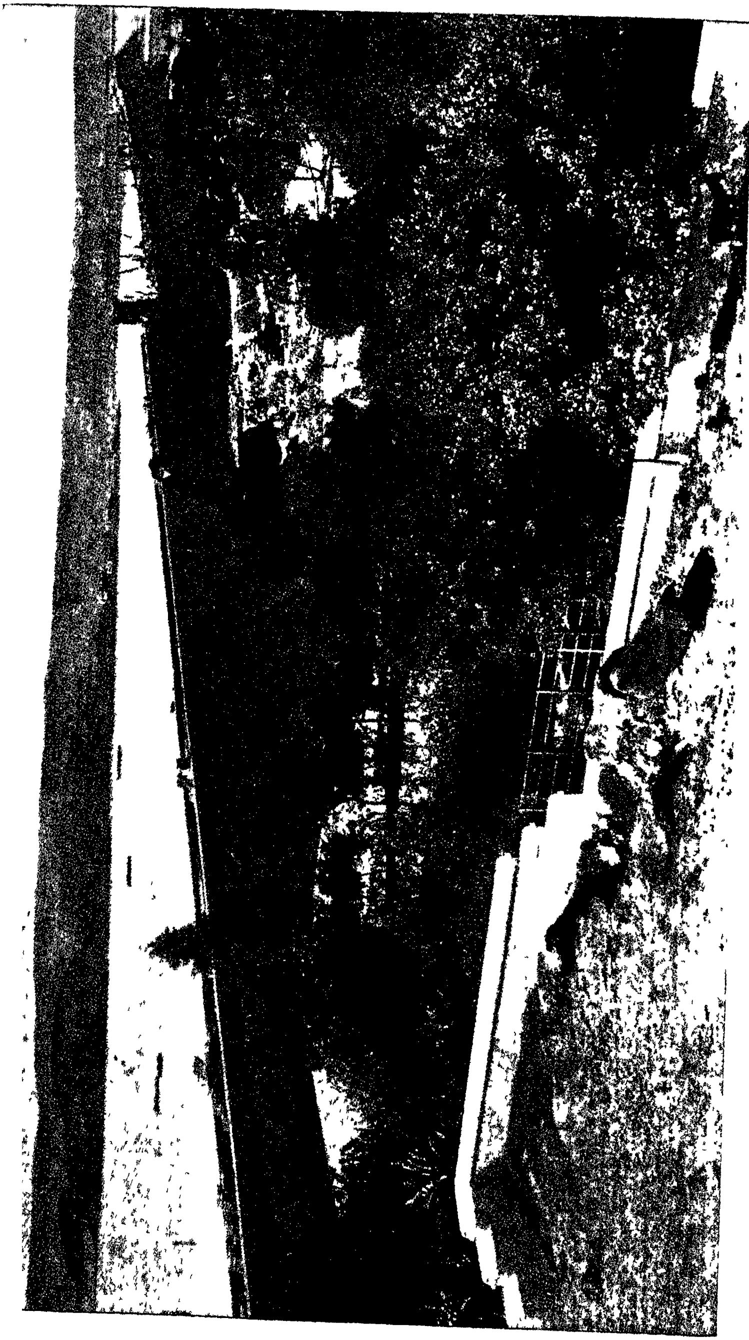


Fig 33. View of Assuan Dam Showing New Work in Progress

49. Cross River and Ashokan Dams. Two further sections are given in Figs. 34 and 35, the first of the Cross River dam, and the second of the Ashokan dam in New York. Both are of unusually thick dimensions near the crest, this being specially provided to enable the dams to resist the impact of floating ice. These profiles are left to be analyzed by the student. The Ashokan dam is provided with a vertical line of porous blocks connected with two inspection galleries. This is a German innovation, which enables any

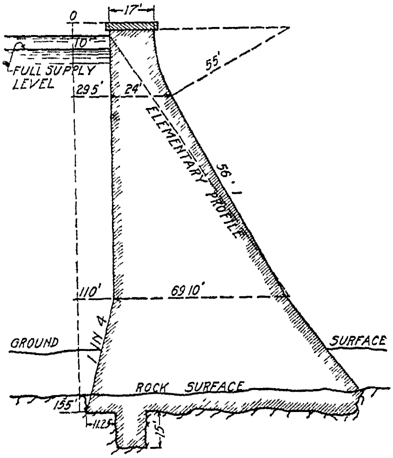


Fig 34 Profile of Cross River Dam

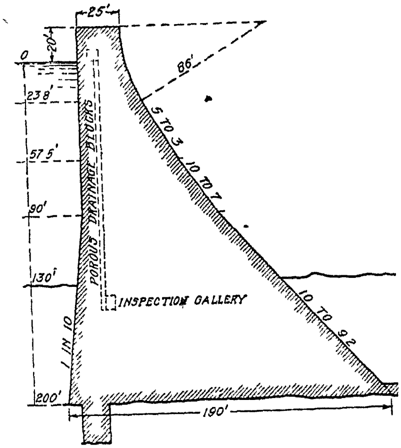


Fig 35 Profile of Ashokan Dam

leakage through the wall to be drained off, thereby guarding against hydrostatic uplift. This refinement is now frequently adopted.

50. Burrin Juick Dam. The Burrin Juick dam in Australia, Fig. 36, which is generally termed "Barren Jack", is a close copy of the Roosevelt dam, Fig. 25, and is a further corroboration of the excellence of that profile. It is built across the Murrumbidgee River in New South Wales not far from the new Federal Capital. Its length is 784 feet on the crest, the maximum height being 240 feet. The fore batter is 3 vertical to 2 horizontal, and the back batter 20 vertical to 1 horizontal, both identical with those adopted in the Roosevelt dam; the crest width is 18 feet. It is built on a curve to a radius of 1200 feet. This dam will impound 785,000 acre-feet. The material of which the dam is composed is crushed sandstone in cement mortar with a plentiful sprinkling of large "plums" of granite. The ultimate resistance of specimen cubes

was 180 "long" tons, per square foot; the high factor of safety of 12 was adopted, the usual being 8 to 10. The maximum allowable stress will, therefore, reduce to 15 "long" tons = 16.8 American short tons.

With regard to the maximum stresses, for *Reservoir Full*, $N=$ 16,100, equivalent to 1210 tons, and q scales about 15 feet, conse-

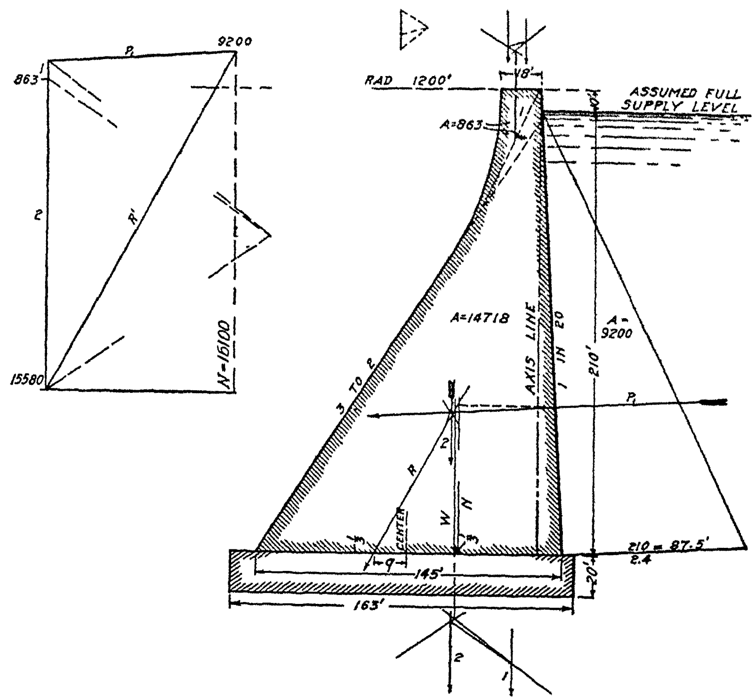


Fig 36 Analytical Diagram Showing Profile of Burrin Juick Dam in Australia

quently m comes to 1.62, and $s=\frac{mN}{b}=\frac{1.62\times1210}{145}=13.5$ tons, and $\frac{P}{b}=\frac{690}{145}=4.8$ tons. Whence

$$c=\frac{s_1}{2}+\sqrt{\frac{s^2}{4}+s_s{}^2}=\frac{13.5}{2}+\sqrt{\frac{(13.5)^2}{4}+(4.8)^2}=15\text{ tons}$$

For Reservoir Empty, $W=15{,}580$ feet or 1170 tons, $q=24$, $m=2$

$$\therefore \qquad s=\frac{mW}{b}=\frac{2\times1170}{145}=16\text{ tons, nearly}$$

The above proves that the stress (R.E.) is greater than that of (R.F.). Probably allowance was made for masses of porous filling lying at the rear of the dam, which would cause N and W to be shifted forward and so equalize the pressure. It will be noticed that the incidence of N, the vertical component (R.E.) falls exactly at the edge of the middle third, a condition evidently observed in the design of the base width.

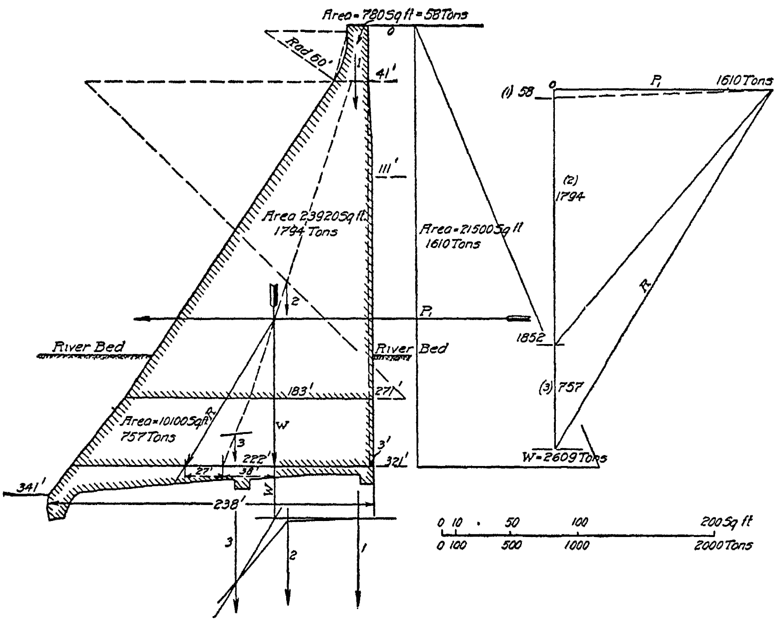


Fig. 37. Profile of Arrow Rock Dam, Idaho, Showing Incidence of Centers of Pressure on Base

The dam is provided with two by-washes 400 feet wide; the reservoir will be tapped by a tunnel 14×13 feet, the entrance sluices of which will be worked from a valve tower upstream, a similar arrangement to that in the Roosevelt dam. It is interesting to note that an American engineer has been put in charge of the construction of this immense work by the Commonwealth Government.

51. Arrow Rock Dam. The highest dam in the world now just completed (1915) is the Arrow Rock on the Boise, Idaho, project, a U. S. reclamation work. From the crest to the base the fore curtain is 351 feet. A graphical analysis of the stress in the

base is given in Fig. 37. For *Reservoir Empty*, $W=2609$ tons, and q measures 38 feet; therefore $m = 1+\frac{228}{222} = 2$ nearly and $s=\frac{2W}{b}=2\times\frac{2609}{222}=23.5$ tons. For *Reservoir Full*, $q=27$, and $m=1+\frac{162}{222}=\frac{1.73\times2609}{222}=20.2$ tons. $s_s=\frac{P}{b}=\frac{1610}{222}=7.3$ tons, and $c=\frac{20.2}{2}+\sqrt{\frac{(20.2)^2}{4}+(7.3)^2}=22.6$ tons. These values are, of course, but approximate.

Thus the compressive stresses (R.F.) and (R.E.) are practically equal, and the incidence of W and also of N is close to the edge of

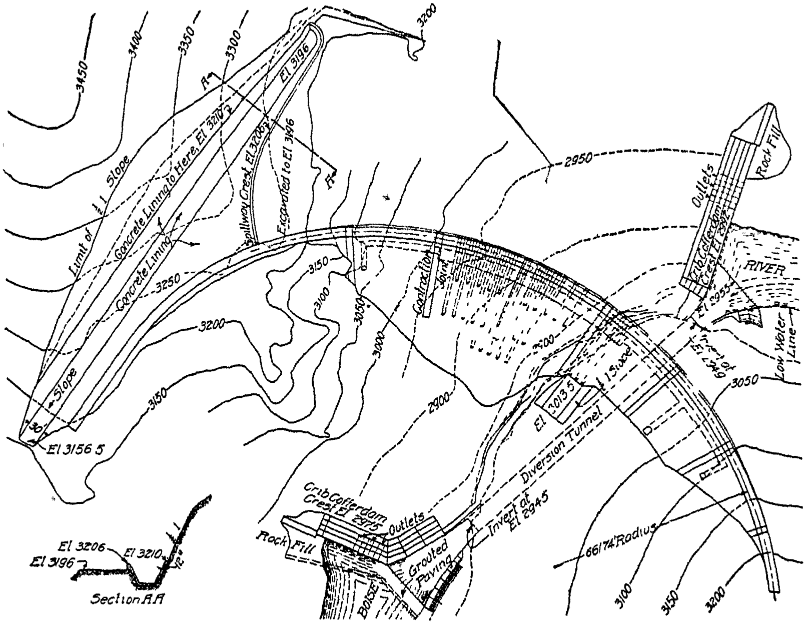


Fig. 38. Location Plan of Arrow Rock Dam
Courtesy of "Engineering Record"

the middle third. The dam is built on a radius of 661 feet at the crest. The high stresses allowed are remarkable, as the design is on the gravity principle, arch action being ignored. The curvature doubtless adds considerably to safety and undoubtedly tends to reduce the compressive stresses by an indeterminate but substantial amount. It is evident that formula (10) has been applied to the

design. Reference to Figs. 38 and 39 will show that the dam is divided into several vertical sections by contraction joints. It is also provided with inspection galleries in the interior and vertical weeper drains 10 feet apart. These intercept any possible seepage, which is carried to a sump and pumped out. These precautions are

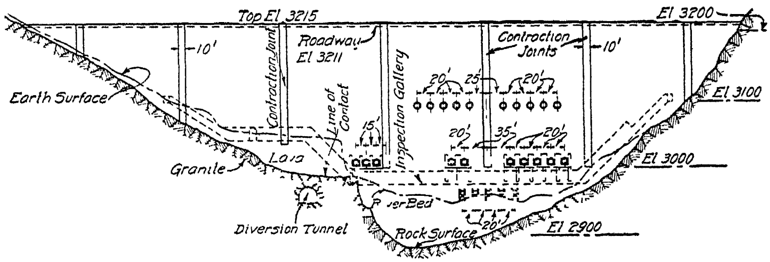


Fig. 39. Elevation of Arrow Rock Dam

to guard against hydrostatic uplift. The simplicity of the outline, resembling that of the Roosevelt dam, is remarkable.

SPECIAL FOUNDATIONS

52. Dams Not Always on Rock. Dams are not always founded on impervious rock but sometimes, when of low height, are founded on boulders, gravel, or sand. These materials when restrained from spreading, and with proper arrangements to take care of subpercolation, are superior to clay, which latter is always a treacherous material to deal with. When water penetrates underneath the base of a dam, it causes hydrostatic uplift, which materially reduces the effective weight of the structure. Fig. 40 represents a wall resting on a pervious stratum and upholding water. The water has ingress into the substratum and the upward pressure it will exert at *c* against the base of the wall will be that due to its depth, in this case 30 feet. Now the point of egress of the percolation will be at *b*, and, as in the case of a pipe discharging in the open, pressure is nil at that point; consequently the uplift area below the base will be a triangle whose area equals $\frac{H \times b}{2}$. The diagram, Fig. 40, shows the combinations of the horizontal water pressure P with the hydrostatic uplift V and with the weight of the wall W. P is first combined with V, R_1 resulting, whose direction is upward. R_1 is then

combined with W, R_2 being their resultant. The conditions without uplift are also shown by the dotted line drawn parallel to dc in Fig. 40. The line ab is termed the hydraulic gradient; it is also the piezometric line, i.e., a line connecting water levels in piezometer tubes, were such inserted.

Fig. 41 shows the same result produced on the assumption that the portion of the wall situated below the piezometric line is reduced in weight by an equal volume of water, i.e., the s.g. of this part may be assumed reduced by unity, i.e., from 2.4 to 1.4. The wall is

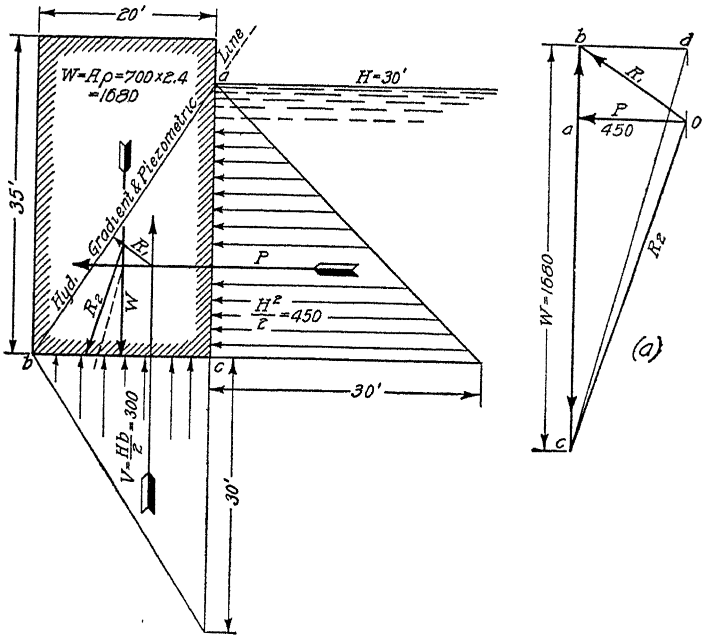


Fig 40 Effect of Uplift on Dam Shown Graphically

thus divided diagonally into two parts, one of s.g. 2.4 and the other of s.g. 1.4. The combination of 1+2 with P is identical in result with that shown in Fig. 40. In the subsequent section, dealing with "Submerged Weirs on Sand", this matter of reduction in weight due to flotation is frequently referred to.

53. Aprons Affect Uplift. Fig. 42 is further illustrative of the principle involved in dams with porous foundations. The pentagonal profile abc, is of sufficient base width, provided hydrostatic uplift is absent. Supposing the foundation to be porous, the area

of uplift will be a_1bc, in which ba_1, equals ab. This area is equal to abc, consequently practically the whole of the profile lies below the hydraulic gradient, may be considered as submerged, and hence loses weight; its s.g. can thus be assumed as reduced by unity, i.e., from ρ to $\rho-1$. The correct base width will then be found by making $b=\frac{H}{\sqrt{\rho-1}}$ instead of $\frac{H}{\sqrt{\rho}}$ The new profile will then be adb; the base width having been thus extended, the uplift is likewise increased in the same proportion. Now supposing an impervious apron to be built in front of the toe as must be the case with an overfall dam; then the area of uplift becomes ba_1e, and the piezo-

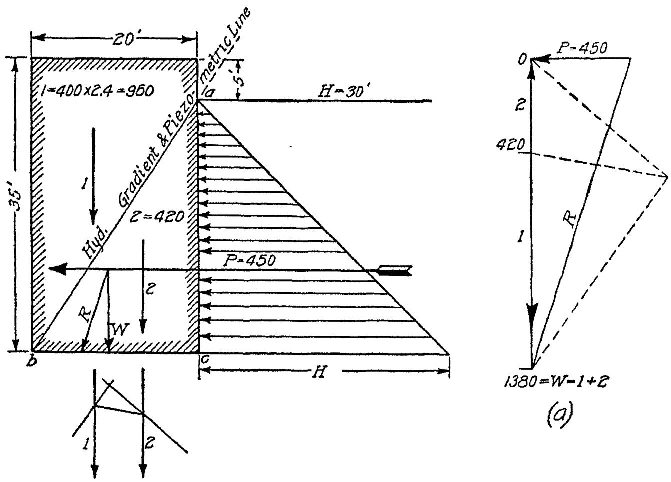


Fig 41. Diagram Showing Identical Result If Weight Is Considered Reduced Due to Submersion

metric line and hydraulic gradient, which in all these cases happen to be one and the same line, is ae. Under these circumstances the comparatively thin apron is subjected to very considerable uplift and will blow up unless sufficiently thick to resist the hydrostatic pressure. The low water, or free outlet level, is assumed to be at the level e, consequently the fore apron lies above this level and is considered as free from flotation due to immersion.

54. Rear Aprons Decrease Uplift. Another case will now be examined. In Fig. 42 suppose the fore apron removed and a rear apron substituted. In this case the point of ingress of the percolating water is thrown back from b to b' the hydraulic gradient is $a'c$,

the triangle of hydrostatic uplift is $b'a_2c$. This uplift from b' to b is more than neutralized by the rectangle of water $a'abb'$, which overlies the rear apron; the latter is therefore not subject to any uplift and, owing to its location, is generally free from erosion by moving water, consequently it can be made of clay, which in this position is water-tight as concrete masonry. A glance at Fig. 42 will demonstrate at once the great reduction of uplift against the base of the wall effected by the expedient of a rear apron, the uplift being reduced from a_1bc to fbc, more than one-half. Thus a rear

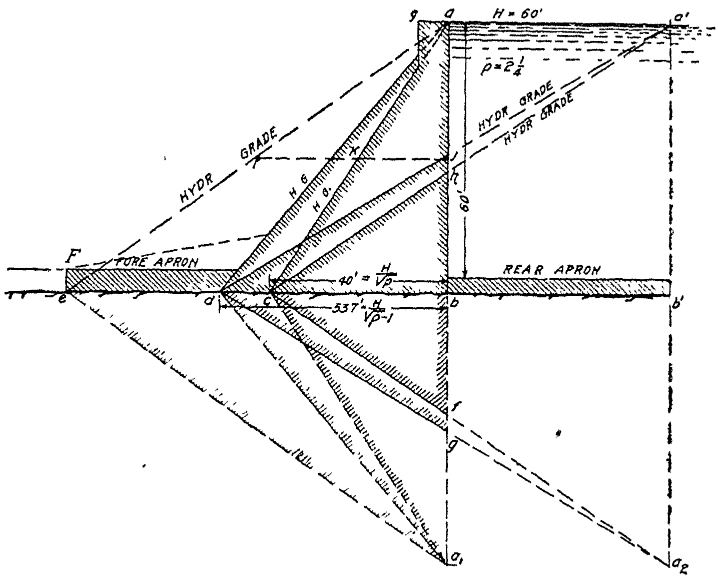


Fig. 42 Diagram Showing Uplift with and without Fore and Rear Aprons

apron is a sure remedy for uplift while the fore apron, if solid, should be made as short as possible, or else should be formed of open work, as heavy slabs with open joints. In the rear of overfall dams stanching clay is often deposited by natural process, thus forming an effective rear apron. Many works owe their security to this fact although it often passes unrecognized.

55. Rock Below Gravel. Fig. 43 represents a dam founded on a stratum of pervious material beneath which is solid rock. A fore curtain wall is shown carried down to the impervious rock. The conditions now are worse than those resulting from the imper-

vious fore apron in Fig. 42 as the hydraulic gradient and piezometric line are now horizontal. The reduced area of vertical hydrostatic pressure is 1066 against which the wall can only furnish 1200; there is, therefore, an effective area of only 134 to resist a water pressure at the rear of 800, consequently the wall must fail by sliding or overturning as the graphical stress lines clearly prove. The proper position of a diaphragm curtain wall is at the heel, not at the toe of the dam; in this location it will effectively prevent all uplift. In the case where an impervious stratum does not occur at a reasonable depth the remedy is to provide a long rear apron which will reduce hydrostatic uplift to as small a value as may be desired, or else a combination of a vertical diaphragm with a horizontal apron can be

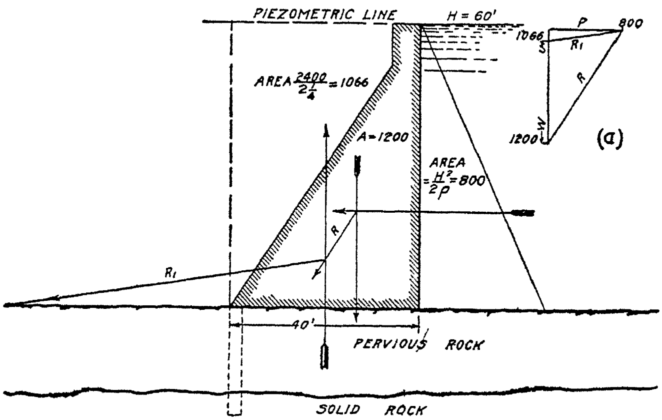


Fig. 43. Effect of Impervious Fore Curtain Wall on Uplift

used. In many cases a portion only of the required rear apron need be provided artificially. With proper precautionary measures the deposit of the remaining length of unfinished apron can safely be left for the river to perform by silt deposit, if time can be afforded for the purpose.

56. Gravity Dam Reinforced against Ice Pressure. This section will be concluded with a recent example of a gravity dam reinforced against ice pressure, which is given in Fig. 44, viz, that of the St. Maurice River dam situated in the Province of Quebec. The ice pressure is taken as 25 tons per foot run, acting at a level corresponding to the crest of the spillway, which latter is shown in Fig. 58. The profile of Fig. 44 is pentagonal, the crest has been given

the abnormal width of 20 feet, while the base is $\frac{3}{4}$ of the height, which is about the requirement, were ice pressure not considered. The horizontal ice pressure, in addition to that of the water upheld, will cause the line of pressure to fall well outside the middle third, thus producing tension in the masonry at the rear of the section. To obviate this, the back of the wall is reinforced with steel rods to the extent of $1\frac{1}{2}$ square inches per lineal foot of the dam. If the safe tensile strength of steel be taken at the usual figure of 16,000

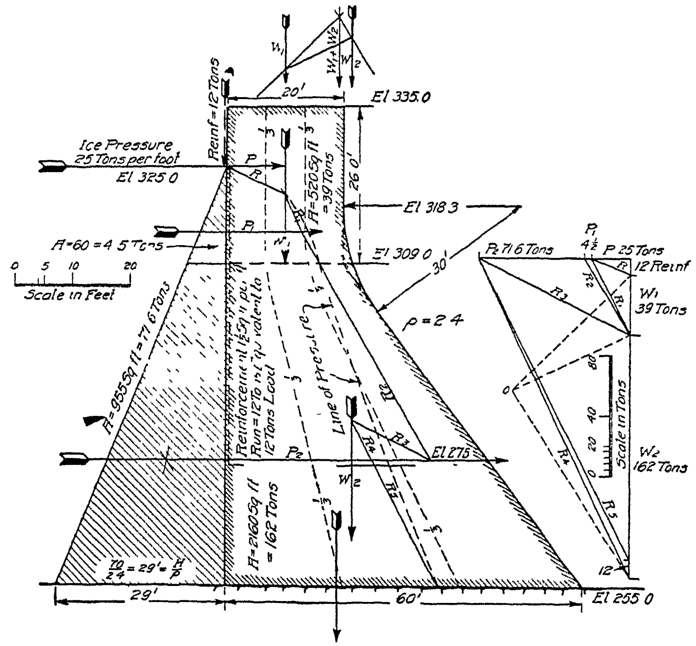


Fig 44 Profile of Saint Maurice River Dam at Quebec

pounds, or 8 tons per square inch, the pull exerted by the reinforcement against overturning will be 12 tons per foot run. This force can be considered as equivalent to a load of like amount applied at the back of the wall, as shown in the figure. The section of the dam is divided into two parts at *El* 309 and the incidence of the resultant pressure at this level and at the base is graphically obtained. The line of pressure connecting these points is drawn on the profile. The line falls outside the middle third in the upper half of the section and within at the base, the inference being that the

section would be improved by conversion into a trapezoidal outline with a narrower crest and with some reinforcement introduced as has been done in the spillway section, shown in Fig. 58.

It will be noticed that the reinforcement stops short at *El* 275. This is allowed for by assuming the imposed load of 12 tons removed at the base of the load line in the force polygon. The line R_5 starting from the intersection of R_4 with a horizontal through *El* 275.0 is the final resultant at the base. This example is most instructive as illustrating the combination of reinforcement with a gravity section in caring for ice pressure, thus obviating the undue enlargement of the profile.

GRAVITY OVERFALL DAMS OR WEIRS

57. Characteristics of Overfalls. When water overflows the crest of a dam it is termed an overfall dam or weir, and some modification in the design of the section generally becomes necessary. Not only that, but the kinetic effect of the falling water has to be provided for by the construction of an apron or floor which in many cases forms by far the most important part of the general design. This is so pronounced in the case of dwarf diversion weirs over wide sandy river beds, that the weir itself forms but an insignificant part of the whole section. The treatment of submerged weirs with aprons, will be given elsewhere. At present the section of the weir wall alone will be dealt with.

Typical Section. Fig. 45 is a typical section of a trapezoidal weir wall with water passing over the crest. The height of the crest as before, will be designated by H, that of reservoir level above crest by d, and that of river below by D. The total height of the upper still water level, will therefore, be $H+d$.

The depth of water passing over the crest should be measured some distance upstream from the overfall just above where the break takes place; the actual depth over the crest is less by reason of the velocity of the overfall being always greater than that of approach. This assumes dead water, as in a reservoir, in the upper reach. On a river or canal, however, the water is in motion and has a velocity of approach, which increases the discharge. In order to allow for this, the head (h) corresponding to this velocity, or $\frac{V^2}{2g}$

multiplied by 1.5 to allow for impact, or $h=.0233V^2$, should be added to the reservoir level. Thus supposing the mean velocity of the river in flood to be 10 feet per second $100\times.0233$ or 2.3 feet would have to be added to the actual depth, the total being 15 feet in Fig. 45. The triangle of water pressure will have its apex at the surface, and its base will, for the reasons given previously, be taken as the depth divided by the specific gravity of the material of the wall. The triangle of water pressure will

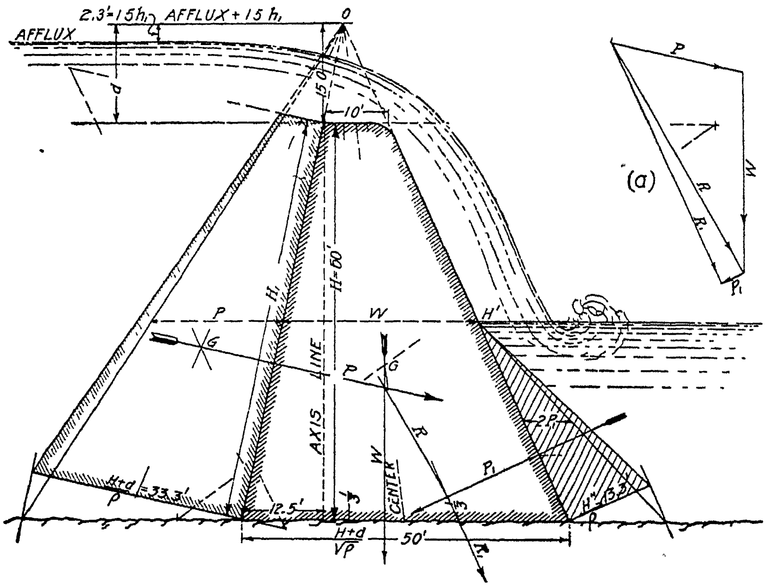


Fig 45 Typical Section of Trapezoidal Weir Wall

be truncated at the crest of the overfall. The water pressure acting against the back of the wall will thus be represented by a trapezoid, not a triangle, whose base width is $\frac{H+d}{\rho}$ and its top width at crest level $\frac{d}{\rho}$. Its area therefore (back vertical) will be $\left(\frac{H+d}{\rho}+\frac{d}{\rho}\right)\frac{H}{2}$. If the back is inclined the side of the trapezoid becomes H_1. The general formula is therefore

$$A=(H \text{ or } H_1)\times\frac{(H+2d)}{2\rho} \qquad \mathbf{(15)}$$

H_1 being the inclined length of the back of the wall. The vertical distance of its point of application above the base according to formula (**5**) page 19 is $h=\frac{H}{3}\left(\frac{H+3d}{H+2d}\right)$ and will be the same whether the back is vertical or inclined.

58. Approximate Base Width. With regard to the drop wall itself, owing to the overfall of water and possible impact of floating timber, ice, or other heavy bodies, a wide crest is a necessity. A further strengthening is effected by adopting the trapezoidal profile. The ordinary approximate rule for the base width of a trapezoidal weir wall will be either

$$b=\frac{(H+d)}{\sqrt{\rho}} \qquad (\mathbf{16})$$

or

$$b=\frac{(H+.6d)}{\sqrt{\rho}} \qquad (\mathbf{16a})$$

The correctness of either will depend on various considerations, such as the value of d, the depth of the overfall, that of h_1 or velocity head and also of D, the depth of the tail water; the inclination given to the back, and lastly, whether the weir wall is founded on a porous material and is consequently subject to loss of weight from uplift. Hence the above formulas may be considered as approximate only and the base width thus obtained subject to correction, which is easiest studied by the graphical process of drawing the resultant on to the base, ascertaining its position relative to the middle third boundary.

59. Approximate Crest Width. With reference to crest width, it may be considered to vary from

$$k=\sqrt{H+d} \qquad (\mathbf{17})$$

to

$$k=\sqrt{H}+\sqrt{d} \qquad (\mathbf{18})$$

the former gives a width sufficient for canal, or reservoir waste weir walls, but the latter is more suitable for river weirs, and is quite so when the weir wall is submerged or drowned.

In many cases, however, the necessity of providing space for falling shutters or for cross traffic during times when the weir is not

acting, renders obligatory the provision of an even wider crest. With a moderate width, a trapezoidal outline has to be adopted, in order to give the requisite stability to the section. This is formed by joining the edge of the crest to the toe of the base by a straight line, the base width of $\frac{(H+d)}{\sqrt{\rho}}$ being adopted, as shown in Fig. 45. When the crest width exceeds the dimensions given in formula (18), the face should drop vertically till it meets the hypothenuse of the elementary profile, as is the case with the pentagonal profile of dams. An example of this is given in Fig. 52 of the Dhukwa weir. The tentative section thus outlined should be tested by graphical process and if necessary the base width altered to conform with the theory of the middle third.

In Fig. 45 is given a diagram of a trapezoidal weir 60 feet high with $d = 15$ feet. According to formula (17) the crest width should be $\sqrt{75} = 8.7$ feet, and according to (18), $7.74 + 3.87 = 11.6$. An average of 10 feet has been adopted, which also equals $\frac{d.}{\rho}$ The profile therefore, exactly corresponds with the elementary triangle canted forward and truncated at the overfall crest.

60. Graphical Process. In graphical diagrams, as has already been explained, wherever possible half widths of pressure areas are taken off with the compasses to form load lines, thus avoiding the arithmetical process of measuring and calculating the areas of the several trapezoids or triangles, which is always liable to error. There are, however, in this case, three areas, one of which, that of the reverse water pressure, has an altitude of only half of the others. This difficulty is overcome by dividing its half width by 2. If one height is not an exact multiple, as this is of H, a fractional value given to the representing line in the polygon will often be found to obviate the necessity of having to revert to superficial measures. The application of the reverse pressure P_1 here exhibited is similar to that shown in Fig. 16; it has to be combined with R, which latter is obtained by the usual process. This combination is effected in the force polygon by drawing a line P_1 equal to the representative area, or half width of the back pressure, in a reverse direction to P. The closing line R_1 is then the final resultant. On the profile itself the force line P_1 is continued through its center of gravity till it

intersects R, from which point R_1 is drawn to the base. If this portion of the face of the weir is very flat, as is sometimes the case, P_1 may be so deflected as to intersect R below the base altogether as is shown in Fig. 50. In such event, R_1 is drawn upward instead of downward to intersect the base. The effect of P_1 is to throw the resultant R_1 farther inward but not to any great extent. It improves the angular direction of R, however.

Reverse Pressure. A dam is usually, but not invariably, exempt from the effect of reverse pressure. This reverse water pressure is generally, as in this case, favorable to the stability of the weir, but there are cases when its action is either too slight to be of service or is even detrimental. This occurs when the face of the weir wall is much inclined, which points to the equiangular profile being most suitable. An example illustrative of the above remarks is given later in Fig. 50 of the Folsam dam.

As the moments of the horizontal pressure of water on either side of the weir wall vary almost with the cubes of their height, it is evident that a comparatively low depth of tail water will have but small influence and may well be neglected. When a vertical back is adopted, the slope is all given to the face; by which the normal reverse water pressure is given a downward inclination that reduces its capacity for helping the wall.

61. Pressures Affected by Varying Water Level. Calculations of the depths of water passing over the weir or rather the height of reservoir level above the weir crest, designated by d, and of the corresponding depth D in the tail channel, are often necessary for the purpose of ascertaining what height of water level upstream, or value of d, will produce the greatest effect on the weir wall. In low submerged or drowned weirs, the highest flood level has often the least effect, as at that time the difference of levels above and below the weir are reduced to a minimum. This is graphically shown in Fig. 46, which represents a section of the Narora dwarf weir wall, to which further reference will be made in section 124, Part II. In this profile two resultant pressures, R and R_1, are shown, of which R_1, due to much lower water level of the two stages under comparison, falls nearer the toe of the base.

The Narora weir, the section of the weir wall of which is so insignificant, is built across the Ganges River in Upper India at the

head of the Lower Ganges Canal, Fig. 93. The principal part of this work, which is founded on the river sand, consists not in the low weir wall, although that is ¾ mile long, but in the apron or floor, which has to be of great width, in this case 200 feet.

As will be seen in Fig. 46 the flood level of the Ganges is 16 feet above the floor level, while the afflux, or level of the head water, is two feet higher. The river discharges about 300,000 second-feet when in flood. When full flood occurs, the weir is completely drowned, but from the diagrams it will be seen that the stress on the wall is less when this occurs than when the head water is

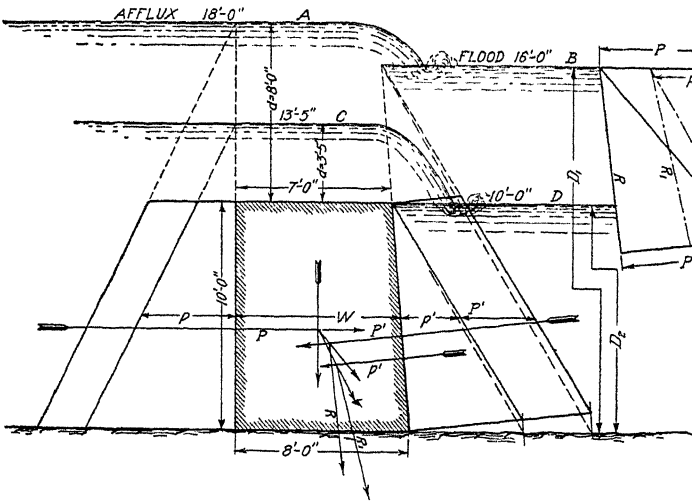


Fig 46 Section of Naroia Dwarf Weir Wall across Ganges River in Upper India

much lower. This result is due to the reverse pressure of the tail water.

The rise of the river water produces, with regard to the stress induced on the weir, three principal situations or "stages" which are enumerated below.

(1) When the head water is at weir crest level; except in cases where a water cushion exists, natural or artificial, the tail channel is empty, and the conditions are those of a dam.

(2) When the level of the tail water lies below weir crest level but above half the height of the weir wall. In this case the reciprocal depth of the head water above crest is found by calculation.

(3) At highest flood level, the difference between the head and tail water is at a minimum. In an unsubmerged weir or overfall dam the greatest stress is generally produced during stage (3). In a submerged weir the greatest stress is produced during stage (2).

62. Moments of Pressure. The moments of the horizontal water pressure on either side of a wall are related to each other in proportion to the cubes of their respective depths. In cases where the wall is overflowed by the water, the triangle of pressure of the latter, as we have seen, is truncated at the weir crest. The moment (M) of this trapezoidal area of pressure will be the product of its area with h, or the product of the expressions in formula (1) and formula (5) as follows:

$$M = H\frac{(H+2d)}{2\rho} \times \frac{H}{3} \times \frac{(H+3d)}{(H+2d)} \times w$$

or

$$M = \frac{H^2 w}{6\rho}\ (H+3d) \qquad \textbf{(19)}$$

That of the opposing tail water will be $M = \frac{D^3 w}{6\rho}$, the difference of these two being the resultant moment. For example, in the case shown in Fig. 46, during stage (1) $H=10$, $D=0$, unbalanced moment $=\frac{10^3 w}{6\rho} = 166.6\frac{w}{\rho}$. In stage (2) $H=10$, $d=3.5$, and $D=10$. Then the unbalanced moment will be $\frac{w}{6\rho}\,[(100 \times 20.5) - 1000] = 175\frac{w}{\rho}$. In stage (3) $H=10$, $D=16$; $d=8$, and $D-H=6$ feet. There will thus be two opposing trapezoids of pressure, and the difference in their moments will be

$$\frac{w(100 \times 34)}{6\rho} - \frac{w(100 \times 28)}{6\rho} = 100\frac{w}{\rho}$$

Thus stage (2) produces the greatest effect, the least being stage (3). In this expression (w) symbolizes, as before, the unit weight of water, per cubic foot or, $\frac{1}{32}$ ton.

In spite of this obvious fact, many weir wall sections have been designed under the erroneous supposition that the overturning moment is greatest when the upper water is at crest level and the tail channel empty, i.e., at a time when the difference of levels above and

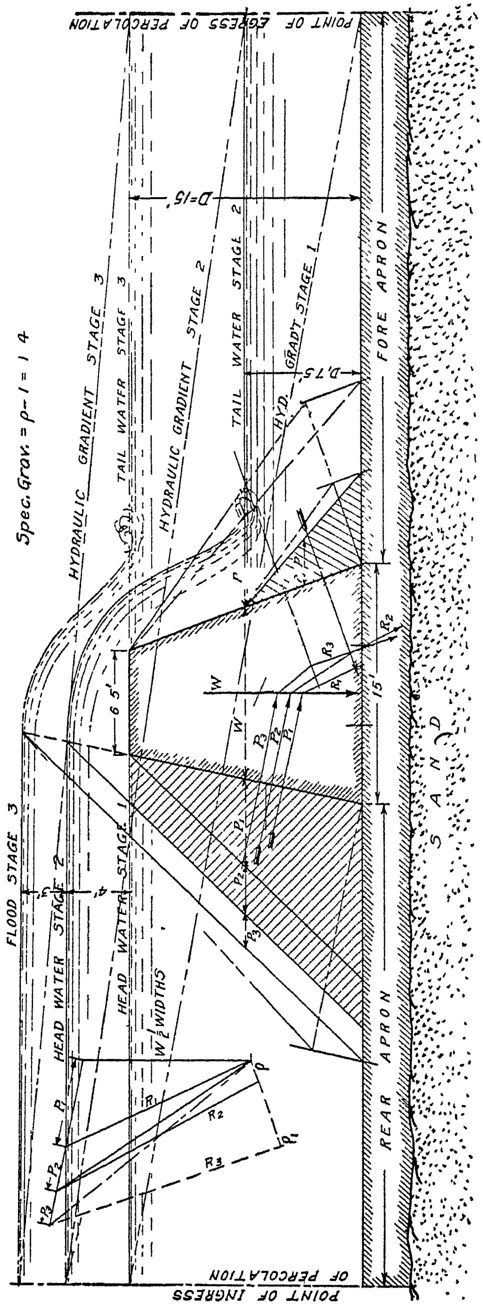


Fig 47. Diagram Showing Analysis of Weir Wall Profile for Three Head-Water Levels

below the weir is at a maximum, or at full flood when the difference is at a minimum.

63. Method of Calculating Depth of Overfall. During the second stage of the river the value of d, the depth of the overfall, will have to be calculated. To effect this the discharge of the river must first be estimated when the surface reaches the crest level of the weir, which is done by use of the formula, $Q = Ac\sqrt{rs}$, given in section 35, page 47 of "Hydraulics, American School of Correspondence", A being the area, equal to d times length of weir (c) Kutter's coefficient, (r) the hydraulic mean radius, and s, the surface grade or slope of the river. The discharge for the whole river should now be divided by the length of the weir crest, the quotient giving the unit discharge, or that per foot run of the weir.

The depth required to pass this discharge with a free overfall is found by use of Francis' formula of $3.33d^{\frac{3}{2}}$ or a modification of it for wide crest weirs for which tables are most useful. See "Hydraulics", section 24, p. 30.

For example, supposing the river discharge with tail water up to crest level is 20 second-feet per foot run of the weir. Then $3.33d^{\frac{3}{2}}=20$. Whence $d^{\frac{3}{2}}=6$ and $d=\sqrt[3]{6^2}=3.3$ feet. This ignores velocity of approach, a rough allowance for which would be to decrease d by (h_1) the velocity head, or by $.0155V^2$.

64. Illustrative Example. Fig. 47 illustrates an assumed case. Here the weir is 15 feet high, 3 stages are shown:

(1) When head water is at crest level;

(2) When tail water is $7\frac{1}{2}$ feet deep, and the reciprocal depth of the head water is assumed as 4 feet; and

(3) With tail water at crest level and head water assumed 7 feet deep above crest.

The three resultants have been worked out graphically. From their location on the base the greatest stress is due to R_2, i.e., stage (2).

The hydraulic gradients of all three stages have been shown with an assumed rear and fore apron on floor. In (1) more than half the weir body lies below the piezometric line, which here corresponds with the hydraulic gradient, while in (2) nearly the whole lies below this line and in (3) entirely so.

Owing to this uplift it is well always to assume the s.g. of a weir wall under these conditions as reduced by immersion to a value of $\rho-1$. In these cases the triangles of water pressure are shown with their bases made $\frac{H}{\rho-1}$, or $\frac{H}{1.4}$, instead of $\frac{H}{2.4}$. Actually, however, the resistance of the weir wall to overturning relative to its base at floor level is not impaired by flotation, but as weight in these cases is a desideratum, the weir wall should be designed as if this were the case. The rear apron is evidently subject to no uplift, but the fore apron is, and its resisting power, i.e., effective weight, is impaired by flotation See section 52 and also the later sections on "Submerged Weirs in Sand", Part II.

65. Examples of Existing Weirs. Some examples of existing weirs will now be given. Fig. 48 is a profile of the LaGrange over-

fall dam at the head of the Modesto and Tuolumne canals, Fig. 49. No less than 13 feet depth of water passes over its crest, 2 feet being

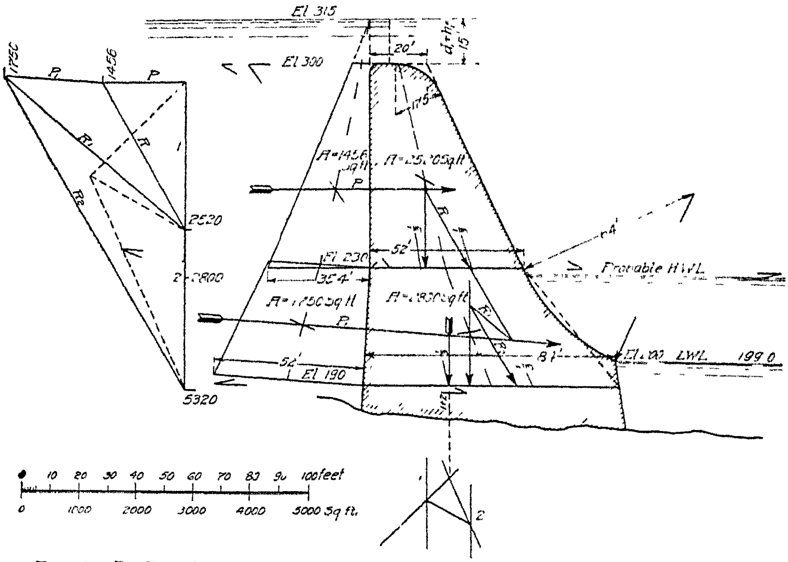


Fig 48. Profile of LaGrange Overfall Dam at Head of Modesto and Tuolumne Canals

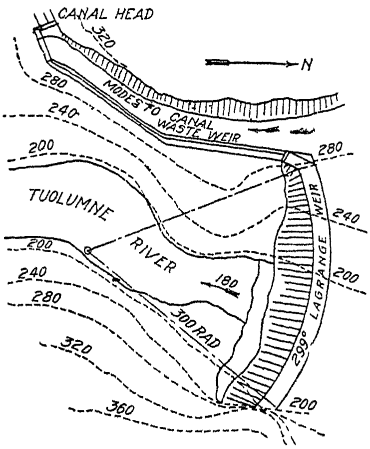


Fig 49. Location Plan of LaGrange Weir

added to allow for velocity of approach. It is built on a curve of 300 feet radius. The graphical analysis of the section shows that the resultants (R.E.) and (R.F.) drawn on the profile fall within the middle third. In this process the reverse pressure due to tail water has been neglected. Its effect will be small.

It is a doubtful point whether the reverse pressure actually exercised is that due to the full depth of the tail water. The overflow causes a disturbance and probably more or less of a vacuum at the toe of the weir wall, besides which the velocity of impact causes a hollow to be formed which must reduce the reverse pressure. In some instances, as in the case of the Granite Reef dam, Fig. 55, the

effective depth of the tail water is assumed as only equal to that of the film of overflow. This appears an exaggerated view. However, in a high overfall dam, the effect of the reverse is often so small that it can well be neglected altogether. In cases where the tail water rises to $\frac{3}{4}$ or more of the height of the dam its effect begins to be considerable, and should be taken into account.

66. Objections to "Ogee" Overfalls. Professional opinion seems now to be veering round in opposition to the "bucket" or curved base of the fore slope which is so pronounced a feature in American overfall dams. Its effects are undoubtedly mischievous, as the destructive velocity of the falling water instead of being reduced as would be the case if it fell direct into a cushion of water, is conserved by the smooth curved surface of the bucket. In the lately constructed Bassano hollow dam (see Figs. 84 and 85, Part II), the action of the bucket is sought to be nullified by the subsequent addition of baffles composed of rectangular masses of concrete fixed on the curved slope. The following remarks in support of this view are excerpted from "The Principles of Irrigation Engineering" by Mr. F. H. Newell, formerly Director of United States Reclamation Service. "Because of the difficulties involved by the standing wave or whirlpool at the lower toe of overflow dams, this type has been made in many cases to depart from the conventional curve and to drop the water more nearly vertically rather than to attempt to shoot it away from the dam in horizontal lines."

67. Folsam Weir. Fig. 50 is of the Folsam weir at the head of the canal of that name. It is remarkable for the great depth of flood water passing over the crest which is stated to be over 30 feet deep. The stress lines have been put on the profile with the object of proving that the reverse pressure of the water, although nearly 40 feet deep has a very small effect. This is due to the flat inclination given to the lower part of the weir, which has the effect of adding a great weight of water on the toe where it is least wanted and thus the salutary effect of the reverse pressure is more than neutralized. The section is not too heavy for requirements, but economy would undoubtedly result if it were canted forward to a nearly equiangular profile, and this applies to all weirs having deep tail water, and to drowned weirs. It will be noted that a wide crest allows but very little consequent reduction in the base width in any case.

The stress diagram in Figs. 50 and 50a are interesting as showing the method of combining the reverse pressures with the ordinary Haessler's diagram of the direct water pressure. The profile is divided into three parts as well as the direct water pressure, whereas the reverse pressure which only extends for the two lower divisions is in two parts. The stress diagrams present no novel features till R_2 is reached. This force on the profile comes in contact with reverse force 1″ before it reaches its objective 3^1. The effect of the reverse

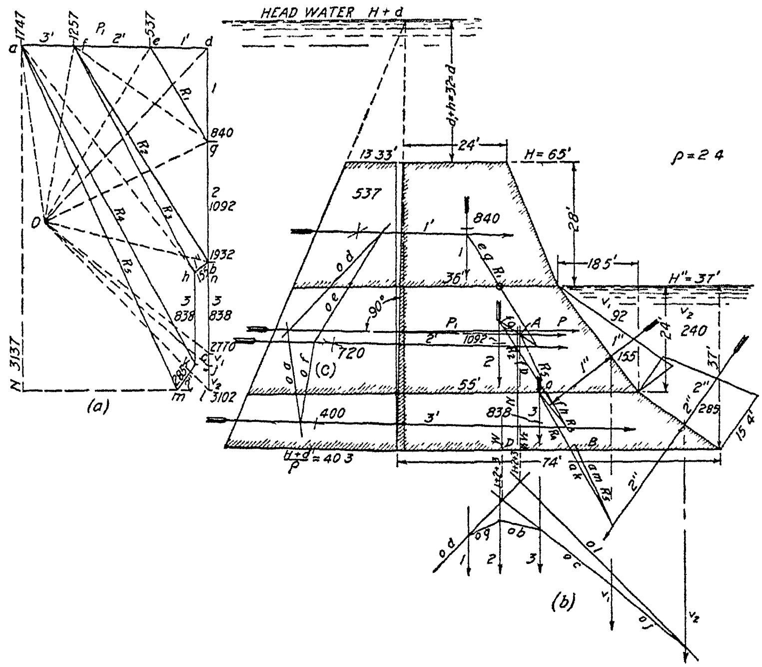


Fig. 50 Graphical Analysis of Folsam Weir

pressure is to deflect the direction of the resultant in the direction of R_3, which latter, as shown in the force polygon, Fig. 50a, is the resultant of 1″, set out from the point *b*, and of R_2. The new resultant R_3 continues till it meets 3^1. The resultant of R_2 and 3^1 is the reverse line drawn upward to meet the vertical force 3, parallel to its reciprocal in Fig. 50a, which is the dotted line joining the termination of 3^1, i.e., (*a*) with that of 1″.

Following the same method the resultant R_4 is next drawn downward to meet 2″, which latter in the force polygon is set out

from the ternination of the vertical (3). The resultant of R_4 and $2''$ is the final R_5. This line is drawn upward on the profile intersecting the base at B. If the reverse pressure were left out of consideration, the force R_2 would continue on to its intersection with 3^1 and thence the reverse recovery line drawn to meet (3) will be parallel to *ba* (not drawn) in the force polygon. This reverse line will intersect the line (3) in the profile almost at the same spot as before.

The final line will be parallel to its reciprocal *ca* (not drawn in Fig. 50a) and will cut the base outside the intersection of R_5. To prevent confusion these lines have not been drawn on; this proves that the effect of the reverse pressure is detrimental to the stability of the wall, except in the matter of the inclination of R_5. If the profile were tilted forward this would not be so. If P_1 the resultant water pressure at the rear of the wall be drawn through the profile to intersect the resultant of all the vertical forces, viz, $1+2+3+v^1+v^2$, this point will be found to be the same as that obtained by producing the final R_5 backwards to meet P_1.

Determination of P_1. To effect this, the position of P_1 has to be found by the following procedure: The load line *db*, Fig. 50a, is continued to l, so as to include the forces 3, v_1, and v_2. The rays *oc*, *oj*, and *ol* are drawn; thus a new force polygon *dol* is formed to which the funicular, Fig. 50b, is made reciprocal. This decides the position of W, or of $1+2+3$, viz, the center of pressure (R. E.) as also that of $W+v_1+v_2$ which latter are the reverse pressure loads. The location of P_1 is found by means of another funicular polygon C derived from the force polygon *oad*, by drawing the rays *oa*, *of*, and *oe*; P_1 is then drawn through the profile intersecting the vertical resultant last mentioned at A. The line AB is then coincident with R_5 on Fig. 50. The vertical line through A is not N, i.e., is not identical with the vertical in Fig. 50, for the reason that N is the resultant of all the vertical forces, whereas the vertical in question is the centroid of pressure of all the vertical force less w_1, the weight of water overlying the rear slope of the wall. The location of N is found by drawing a horizontal P through the intersection of P_1 with a line drawn through the c.g. of the triangle of water pressure w, this will intersect the back continuation of BA at c. A vertical CD through this point will correspond with that marked

N in Fig. 50. The profile, Fig. 51, is a reproduction of that shown Fig. 50 in order to illustrate the analytical method of calculation or that by moments.

68. Analytical Method. The incidence of the resultant R is required to be as ascertained on two bases, one the final base and the other at a level 13 feet higher. The section of the wall as before, is divided into three parts: (1) of area 840 square feet, (2) of 1092, and (3) of 838 square feet. The position of the c.g. of (1) is found by formula (7) to be 15 15 feet distant from a the heel of the base and will be

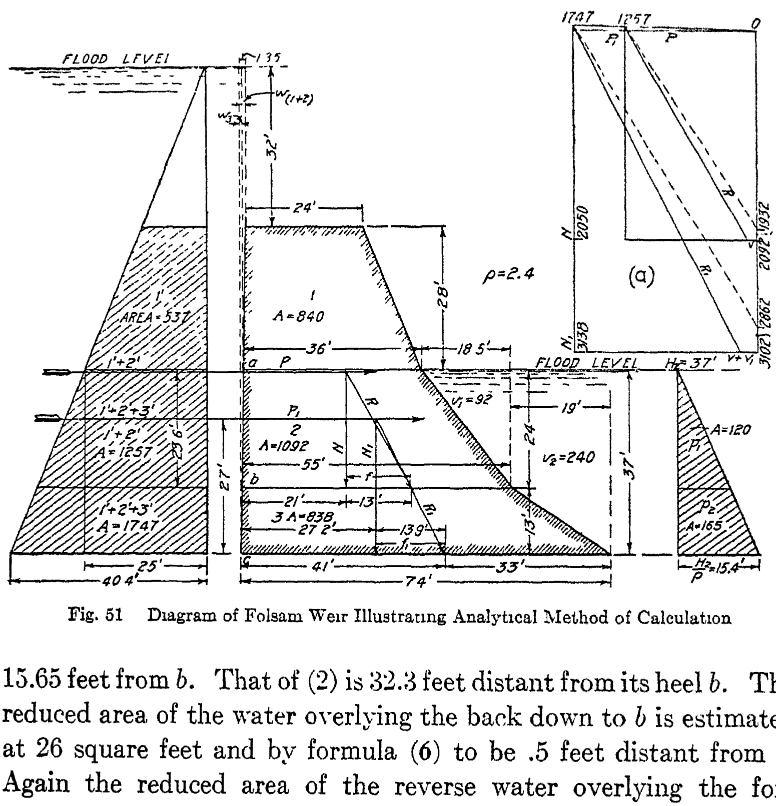


Fig. 51 Diagram of Folsam Weir Illustrating Analytical Method of Calculation

15.65 feet from b. That of (2) is 32.3 feet distant from its heel b. The reduced area of the water overlying the back down to b is estimated at 26 square feet and by formula (6) to be .5 feet distant from b. Again the reduced area of the reverse water overlying the fore slope v_1 is 92 square feet and the distance of its c.g. from b is $55 - \frac{18.5}{3}$ $= 48.8$ feet. The moments of all these vertical forces equated with that of their sum (N) about the point b will give the position of N relative to b.

Thus

$$\begin{array}{lrcl}
(1) & 840\times15.65 & = & 13146\\
(2) & 1092\times23.3 & = & 25443\\
(w) & 26\times\ .5 & = & 13\\
(v_1) & 92\times48.8 & = & 4490\\
\hline
 & 2050\times x & = & 43092 = \text{Moment of } N\\
\therefore & x & = & 21 \text{ feet, nearly}
\end{array}$$

To obtain the distance f between N and R, $f=\dfrac{M_P-M_{p_1}}{N}$. Now the reduced area of $P=1257$ and the height of the c.g. of the trapezoid having its base at b, and its crest level with that of the wall is calculated by formula (**6**), to be 22.1 feet. Again the reduced area of the reverse water pressure triangle p_1 is 120 square feet, the height of its c.g. above base is 8 feet. Consequently:

$$f=\frac{(1257\times22.1)-(120\times8)}{2050}=\frac{26820}{2050}=13 \text{ feet}$$

For the lower base, the statement of moments about c is as follows, v_2 being 240, and its distance 65 feet by formula (**6**).

$$\begin{array}{lrcl}
 & (N)\ 2050\times(21+.3) & = & 43665\\
 & (3)\ \ 838\times32.3 & = & 27067\\
 & (w)\ \ 10\times\ .15 & = & 2\\
 & (v_2)\ \ 240\times65 & = & 15600\\
\hline
\text{Total} & 3138\times x & = & 86334
\end{array}$$

$$\therefore \qquad x-\frac{86334}{3138}=27.4 \text{ feet}$$

Now $f_1=\dfrac{M_{P_1}-M_{p_1+p_2}}{N_1}$, f_1 being the distance between N_1 and R_1. The value of P_1, the trapezoid of water pressure down to the base c, is 1747 square feet and the height of its c.g. by formula (**19**) or (**5**) is 27 feet, that of (p_1+p_2) is 285 square feet and its lever arm $\dfrac{37}{3}=12\frac{1}{3}$ feet. Then

$$f_1=\frac{(1747\times27)-(285\times12\frac{1}{3})}{3138}=\frac{47169-3514}{3138}=\frac{43655}{3138}=13.9 \text{ ft.}$$

The positions of N and N_1 being obtained, the directions of R and R_1 are lines drawn to the intersections of the two verticals

N and N_1 with two lines drawn through the c.g.'s of the trapezoid of pressure reduced by the moment of the reverse pressure, if any, or by $(P-p)$. This area will consist, as shown in the diagram, of a trapezoid superposed on a rectangle; by using formula (**5**) section 1, the positions of the c.g. of the upper trapezoid is found to be 12.58 feet above the base at a, while that of the lower is at half the depth of the rectangle, then by taking moments of these areas about b, the height of the c.g. is found to be 23.6 feet above the base at b, while the height for the larger area $[P_1-(p_1+p_2)]$ down to c is 27 feet.

In the graphical diagram of Fig. 51a the same result would be obtained by reducing the direct pressure by the reverse pressure area. Thus in the force diagram the vertical load line would remain unchanged but the water-pressure load line would be shorter being $P-p$ and $P_1-(p_1+p_2)$, respectively. This would clearly make no difference in the direction of the resultants R and R_1 and would save the two calculations for the c.g.'s of P and P_1.

This weir is provided with a crest shutter in one piece, 150 feet long, which is raised and lowered by hydraulic jacks chambered in the masonry of the crest so that they are covered up by the gate when it falls. This is an excellent arrangement and could be imitated with advantage. The shutter is 5 feet deep. The width at base of lamina 2 of this weir is 55 feet, or very nearly $\frac{H+d}{\sqrt{\rho}}$, formula (**16**).

69. Dhukwa Weir. A very similar work is the Dhukwa weir in India, Fig. 52, which has been recently completed.

This overfall dam is of pentagonal section. Owing to the width of the crest this is obviously the best outline.

The stress resultant lines have been drawn on the profile, which prove the correctness of the base width adopted. The tail water does not rise up to half the height of the weir. Consequently the formula $\frac{H+d}{\sqrt{\rho}}$ is applicable in stage 3. The effect of the tail water is practically nil. According to this formula the base width would be $63\times\frac{2}{3}=42$ feet, which it almost exactly measures—a further demonstration of the correctness of the formula. The crest width should be, according to formula (**18**), $\sqrt{50}+\sqrt{13}=11$ feet. The

width of 17 feet adopted is necessary for the space required to work the collapsible gates. These are of steel, are held in position by struts connected with triggers, and can be released in batches by chains worked from each end. The gates, 8 feet high, are only 10 feet wide. This involves the raising and lowering of 400 gates, the weir crest being 4000 feet long. The arrangement adopted in the Folsam weir of hydraulic jacks operating long gates is far superior. An excellent feature in this design is the subway with occasional side chambers and lighted by openings, the outlook of which is underneath the waterfall, and has the advantage of relieving any vacuum under the falling water.

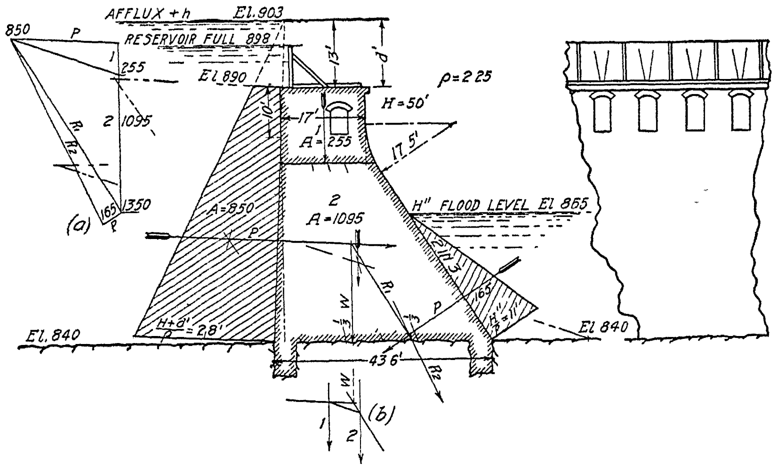


Fig. 52 Graphical Analysis of Profile of Dhukwa Weir in India

The subway could be utilized for pressure pipes and for cross communication, and the system would be most useful in cases where the obstruction of the crest by piers is inadvisable. The weir is 4000 feet long and passes 800,000 second-feet, with a depth of 13 feet. The discharge is, therefore, 200 second-feet per foot run of weir, which is very high. The velocity of the film will be $\frac{200}{13} = 15.4$ feet per second. With a depth of 13 feet still water, the discharge will be by Francis' formula, 156 second-feet per foot run. To produce a discharge of 200 feet per second, the velocity of approach must be about 10 feet per second. This will add 2.3 feet to the actual value

of d, raising it from 13 to 15.3 feet which strictly should have been done in Fig. 52.

70. Mariquina Weir. Another high weir of American design, Fig. 53, is the Mariquina weir in the Philippines. It has the ogee curve more accentuated than in the LaGrange weir. The stress lines have been drawn in, neglecting the effect of the tail water which will be but detrimental. The section is deemed too heavy at the upper part and would also bear canting forward with advantage, but there are probably good reasons why an exceptionally solid

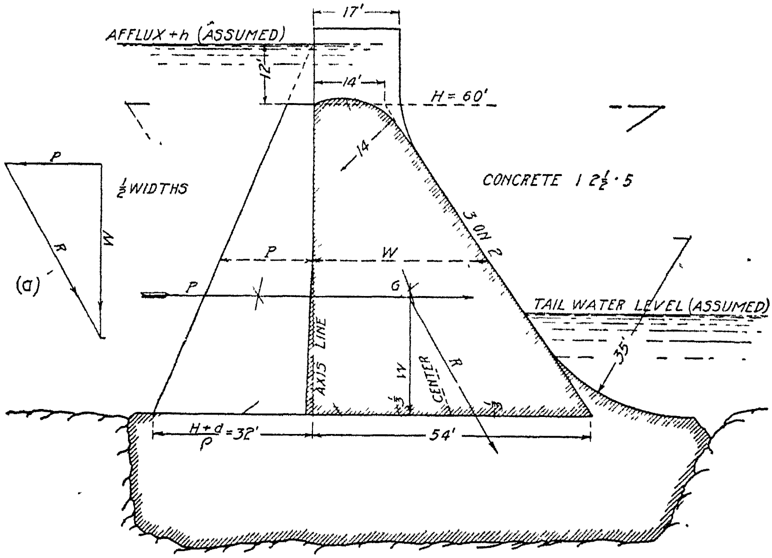


Fig 53 Profile of Mariquina Weir in the Philippines

crest was adopted. The ogee curve also is a matter on which opinion has already been expressed.

71. Granite Reef Weir. The Granite Reef weir over the Salt River, in Arizona, Figs. 54 and 55, is a work subsidiary to the great Roosevelt dam of which mention was previously made.

It is founded partly on rock and partly on boulders and sand overlying rock. The superstructure above the floor level is the same throughout, but the foundations on shallow rock are remarkable as being founded not on the rock itself, but on an interposed cushion of sand. (See Fig. 54.) Reinforced concrete piers, spaced 20 feet apart, were built on the bedrock to a certain height, to clear

all inequalities; these were connected by thin reinforced concrete side walls; the series of boxes thus formed were then filled level with sand, and the dam built thereon. This work was completed in 1908. The portion of the profile below the floor is conjectural. This construction appears to be a bold and commendable novelty. Sand in a confined space is incompressible, and there is no reason why it should not be in like situations. A suggested improvement would be to abandon the piers and form the substructure of two long outer walls only, braced together with rods or old rails encased in concrete. Fig. 55 is the profile on a boulder bed with rock below.

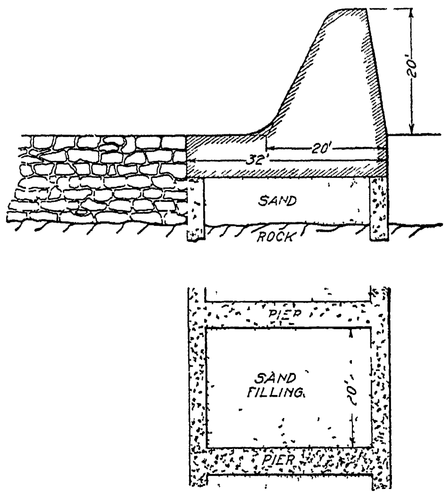


Fig. 54 Section of Granite Reef Weir Showing Sand Cushion Foundations

72. Hydraulic Conditions. The levels of the afflux flood of this river are obtainable so that the stresses can be worked out. In most cases these necessary statistics are wanting. The flood downstream has been given the same depth, 12 feet, as that of the film passing over the crest. This is clearly erroneous. The velocity of the film allowing for 5 feet per second approach, is quite 12 feet per second, that in the river channel could not be much over 5 feet, consequently it would require a depth of $\frac{12\times12}{5}=28$ feet. The dam would thus be quite submerged, which would greatly reduce the stress. As previously stated, the state of maximum stress would probably occur when about half the depth of flood passes over the crest. However, the graphical work to find the incidence of the resultant pressure on the base will be made dependent on the given downstream flood level. After the explanations already given, no special comment is called for except with regard to the reverse water pressure. Here the curved face of the dam is altered into 2 straight lines and the water pressure consists of two forces,

having areas of 17 and 40, respectively, which act through their c.g.'s. Instead of combining each force separately with the result-

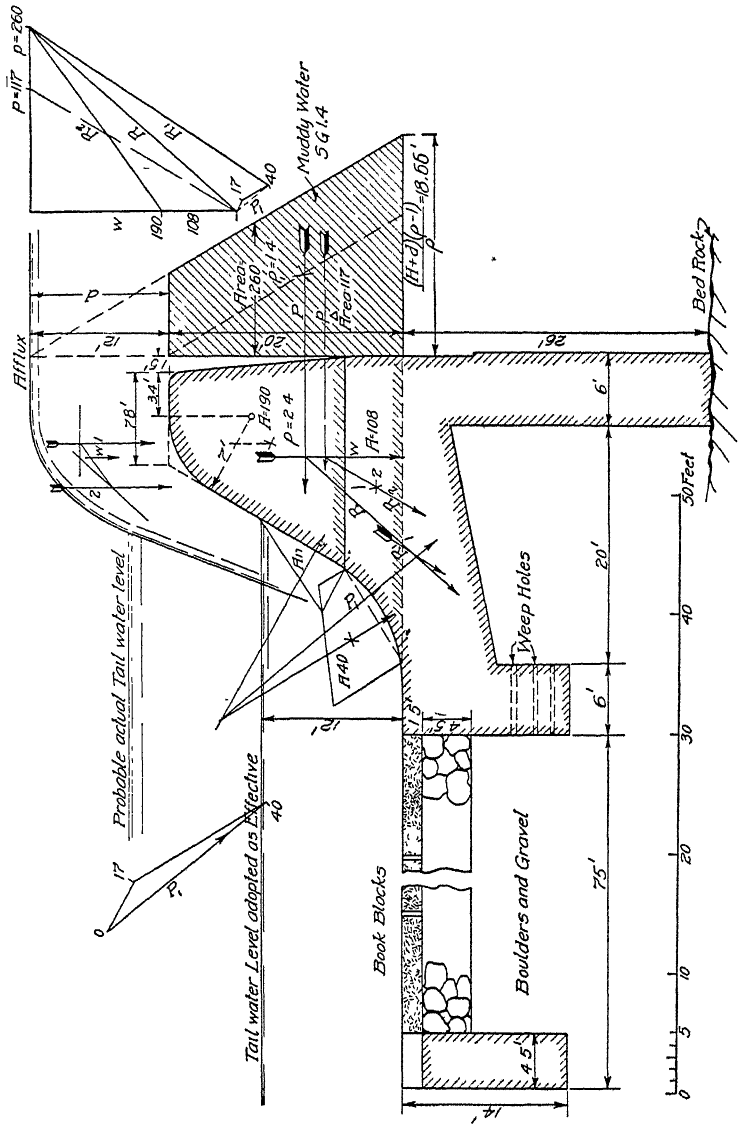


Fig. 55. Graphical Analysis of Profile of Granite Reef Dam over Salt River, Arizona

ant (R) it is more convenient to find their resultant and combine that single force with (R.) This resultant P_1 must pass through the intersection of its two components, thus if their force lines are

run out backward till they intersect, a point in the direction of P_1 is found. P_1 is then drawn parallel to its reciprocal in the force polygon which is also shown on a larger scale at the left of the profile. The final resultant is R_1 which falls just within the middle third of the base. R_2 is the resultant supposing the water to be at crest level only. The water in the river is supposed to have mud in solution with its s.g. 1.4. The base length of the triangle of water pressure will then be $\frac{(H+d)\times(\rho-1)}{\rho}=\frac{32\times1.4}{2.4}=18.66$. The other water-pressure areas are similarly treated. If the rear curtain reaches rock the dam should not be subject to uplift. It could, however, withstand sub-percolation, as the hearth of riprap and boulders will practically form a filter, the material of the river bed being too large to be disintegrated and carried up between the interstices of the book blocks. The effective length of travel would then be 107 feet; add vertical 52 feet, total 159 feet, H being 20 feet, $\frac{L}{H}$ works out to $\frac{160}{20}=8$ which ratio is a liberal allowance for a boulder bed. The fore curtain is wisely provided with weep holes to release any hydrostatic pressure that might otherwise exist underneath the dam. The Granite Reef dam has a hearth, or fore apron of about 80 feet in width. A good empirical rule for the least width for a solid or open work masonry fore apron is the following:

$$L=2H+d \qquad \textbf{(20)}$$

in which H is the height of the permanent weir crest above floor, and d is the depth of flood over crest. In this case $H=20$, $d=12$; least width of floor should then be $40+12=52$ feet. The Bassano dam is 40 feet high with 14 feet flood over crest, the width of hearth according to this formula should be 94 feet, its actual width is 80 feet which is admittedly insufficient. With a low submerged weir, formula (34), Part II, viz, $L=3\sqrt{cH}$, will apply. Beyond the hearth a talus of riprap will generally be required, for which no rule can well be laid down.

73. Nira Weir. Fig. 56 is of the Nira weir, an Indian work. Considering the great depth of the flood waterdown stream, the provision of so high a subsidiary weir is deemed unnecessary, a wa errar cushion of 10 feet being ample, as floor is bed rock. The sectio e river-

the weir wall itself, is considered to be somewhat deficient in base width. Roughly judging, the value of $H+d$, on which the base width is calculated, should include about 3 or 4 feet above crest level. This value of d, it is believed would about represent the height of head water, which would have the greatest effect on the weir. The exact value of d could only be estimated on a knowledge of the bed slope or surface grade of the tail channel. The above estimate would make $(H+d)=36$ feet, and with $\rho=2\frac{1}{4}$, $\frac{H+d}{\sqrt{\rho}}=24$ feet.

The top width, 8.3, is just $\sqrt{H}+\sqrt{d}$, in accordance with the rule given in formula (18).

A section on these lines is shown dotted on the profile. The provision of an 8-foot top width for the subsidiary weir is quite

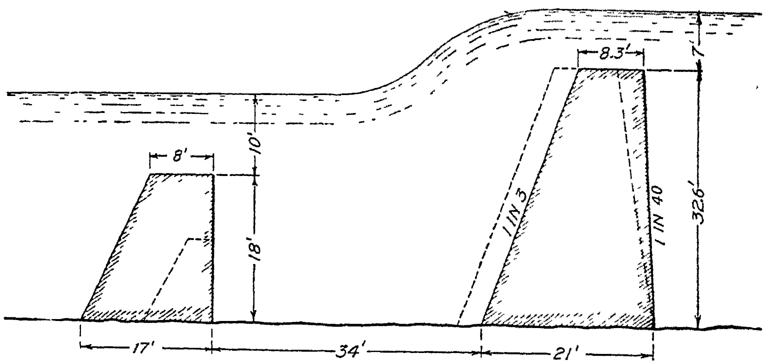


Fig 56 Section of Nira Weir in India Showing Use of Secondary Weir

indefensible, while the base width is made nearly equal to the height, which is also excessive. For purposes of instruction in the principles of design, no medium is so good as the exhibition of plans of actual works combined with a critical view of their excellencies or defects. The former is obtainable from record plans in many technical works, but the latter is almost entirely wanting. Thus an inexperienced reader has no means of forming a just opinion and is liable to blindly follow designs which may be obsolete in form or otherwise open to objection.

74. Castlewood Weir. The Castlewood weir, Fig. 57, is of remarkable construction, being composed of stonework set dry, that enclosed in a casing of rubble masonry. It is doubtful if such a the section is any less expensive than an ordinary gravity section, or

much less than an arched buttress dam of type *C*. Shortly after construction, it showed signs of failure, which was stated to be due to faulty connections with banks of the river; but whatever the cause it had to be reinforced, which was effected by adding a solid bank of earth in the rear, as shown in the figure. This involved lengthening the outlet pipes. In the overfall portion the bank must have been protected with riprap to prevent scouring due to the velocity of the approach current.

75. American Dams on Pervious Foundations. In the United States a very large number of bulkhead and overfall dams and regulating works, up to over 100 feet in height have been built on foun-

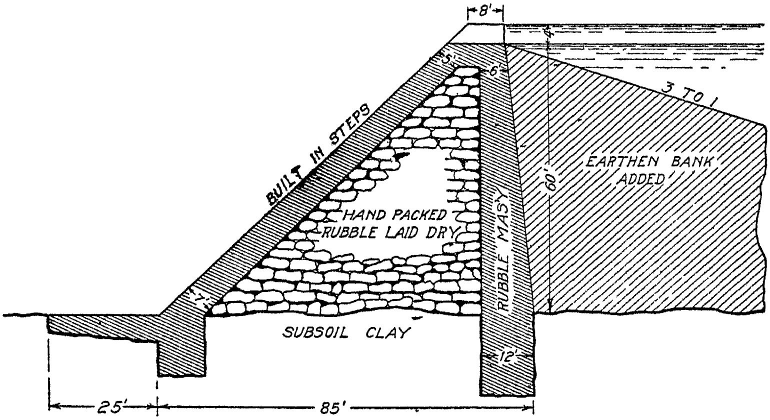


Fig. 57. Section of Castlewood Weir Showing Construction of Stone Work Set Dry, Enclosed in Rubble Masonry

dations other than rock, such as sand, boulders, and clay. Most of these, however, are of the hollow reinforced concrete, or scallop arch types, in which a greater spread for the base is practicable than would be the case with a solid gravity dam. Whenever a core wall is not run down to impervious rock, as was the case in the Granite Reef Overfall dam, Fig. 55, the matter of sub-percolation and uplift require consideration, as is set forth in the sections on "Gravity Dams" and "Submerged Weirs on Sand". If a dam 50 feet high is on sand or sand and boulders, of a quality demanding a high percolation factor of say 10 or 12, it is clear that a very long rear apron and deep rear piling will be necessary for safety.

All rivers bring down silt in suspension. When the overfall dam is a high one with a crest more than 15 or 20 feet above river-

bed level, the deposit that is bound to take place in rear of the obstruction will not be liable to be washed out by the current, and additional light stanching silt will be deposited in the deep pool of comparatively still water that must exist at the rear of every high dam. For a low weir however this does not follow, and if deposit is made it will be of the heavier, coarser sand which is not impermeable.

The difficulty and expense of a long rear apron can be surmounted by the simple expedient of constructing only a portion of it of artificial clay, leaving the rest to be deposited by the river itself. To ensure safety the dam should be constructed and reservoir filled, in two or three stages, with intervals between of sufficient length to allow the natural deposit to take place. Thus only a fraction of the protective apron need be actually constructed. Many works are in existence which owe their safety entirely to the fortunate but unrecognized circumstance of natural deposit having stanched the river bed in their rear, and many failures that have taken place can only be accounted for from want of provision for the safety of the work against underneath scour or piping and also uplift. The author himself once had occasion to report on the failure of a head irrigation work which was designed as if on rock, whereas it was on a pervious foundation of boulders. When it failed the designers had no idea of the real cause, but put it down to a "treacherous river", "ice move", anything but the real reason, of which they were quite ignorant. Had a rear apron of sufficient width been constructed, the work would be standing to this day.

76. Base of Dam and Fore Apron. The fore apron and base of an overfall dam or weir must be of one level throughout its length, if the foundation is of any other material than rock. The foundation core walls may have to vary more or less with the surface of the river bed, which is deep in some places, and shallow in others, but the apron level should be kept at or about low water level throughout. When a horizontal wall as an overfall dam is built across a river bed it obliterates the depressions and channels in it, the discharge over the weir is the same at all points or nearly so, consequently the tendency will be to level the bed downstream by filling the hollows and denuding the higher parts.

Under these conditions it is evidently sheer folly to step up the apron to coincide with the section of the river bed, as the higher

parts of the bed are bound to be in time washed out by the falling water and deposited in the deeper channels, and portions of the dam may easily be undermined. This actually occured in one case.

77. Section of Spillway of St. Maurice River Dam. Fig. 58 is a section of the spillway portion of the reinforced bulkhead gravity dam, illustrated in Fig. 44. Owing to the absence of the heavy crest of Fig. 44, the back of the spillway profile is provided with

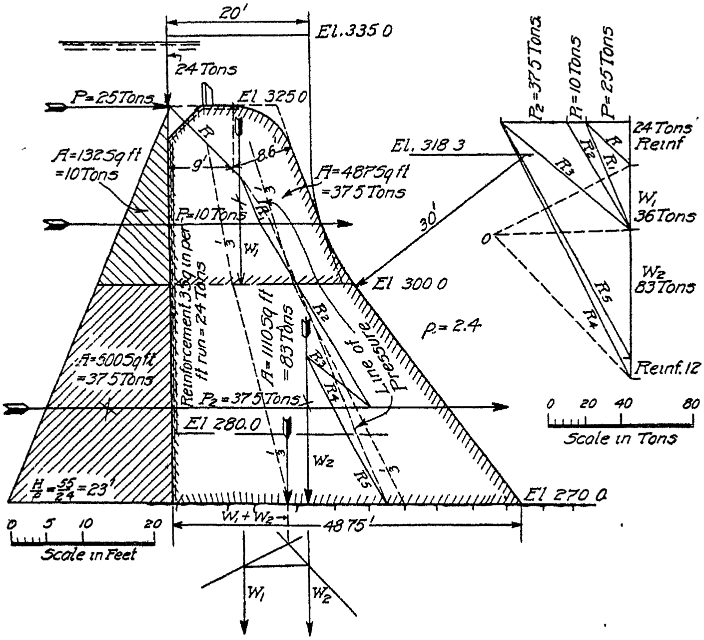
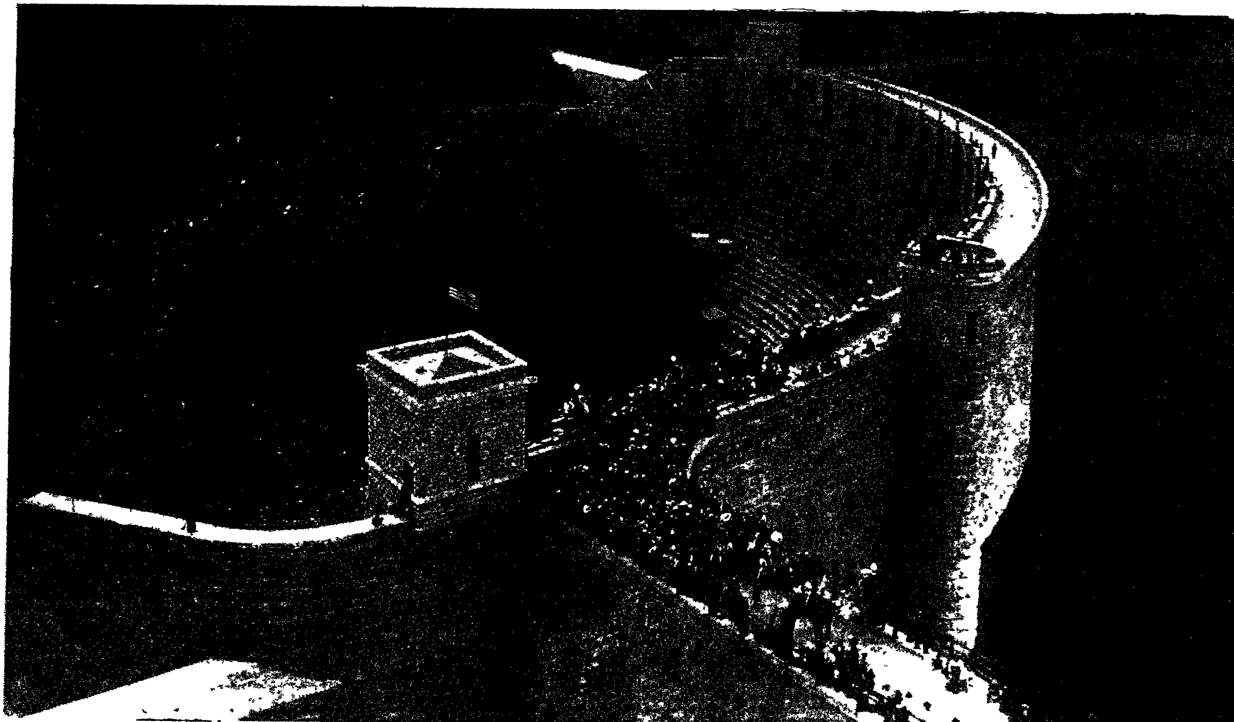


Fig 58 Diagram Showing Profile of Spillway Portion of Saint Maurice River Dam (See Fig 44)

double the amount of reinforcement shown in the former example. One half, viz, 1½ inches, extends right down to the base, while the other half stops short at *El* 280. This is arranged for in the stress diagrams in the same way as explained in section 55, R_5 being the final resultant on the base. The line of pressure falls slightly outside the middle third in the upper half of the section. The effect would be to increase the tension in the reinforcement somewhat above the limit of 8 tons per square inch. The adoption of a trapezoidal profile, would, it is deemed, be an improvement in this case as well as in the former.



ROOSEVELT DAM BEING FORMALLY OPENED BY THEODORE ROOSEVELT
Photo by Brown Brothers, New York City

DAMS AND WEIRS

PART II

ARCHED DAMS

78. **General Characteristics.** In this type, the whole dam, being arched in plan, is supposed to be in the statical condition of an arch under pressure. As, however, the base is immovably fixed to the foundations by the frictional resistance due to the weight of the structure, the lowest portion of the dam cannot possess full freedom of motion nor elasticity, and consequently must act more or less as a gravity dam subject to oblique pressure.

However this may be, experience has conclusively proved that if the profile be designed on the supposition that the whole is an elastic arch, this conflict of stresses near the base can be neglected by the practical man. The probability is that both actions take place, true arch action at the crest, gradually merging into transverse stress near the base; the result being that the safety of the dam is enhanced by the combination of tangential and vertical stresses on two planes.

In this type of structure, the weight of the arch itself is conveyed to the base, producing stress on a horizontal plane, while the water pressure normal to the extrados, radial in direction, is transmitted through the arch rings to the abutments. The pressure is, therefore, distributed along the whole line of contact of the dam with the sides as well as the ground. In a gravity dam, on the other hand, the whole pressure is concentrated on the horizontal base.

Arch Stress. The average unit stress developed by the water pressure is expressed by the formula

$$s_1=\frac{RHw}{b} \qquad \text{"Short" Formula} \quad \mathbf{(21)}$$

$$\therefore \quad b=\frac{RHw}{s_1} \qquad \text{"Short" Formula} \quad \mathbf{(21a)}$$

in which R is the radius of the extrados, sometimes measured to the center of the crest, H the depth of the lamina, b its width, and w the unit weight of water or $\frac{1}{32}$ ton. Into this formula ρ, the specific gravity of the material in the arch, does not enter. This simple formula answers well for all arched dams of moderate base width. When, however, the base width is considerable, as, say, in the case of the Pathfinder dam, the use of a longer formula giving the maximum stress (s) is to be preferred. This formula is derived from the same principle affecting the relations of s and s_1, or of the maximum and average stresses already referred to in Part I on "Gravity Dams". The expression is as follows, r being the radius of the intrados:

$$s = s_1 \frac{2R}{(R+r)} = \frac{RHw}{b} \times \frac{2R}{R+r}$$

or in terms of R and b

$$s = \frac{2Hw}{\frac{b}{R}\left(2 - \frac{b}{R}\right)} \qquad \text{"Long" Formula} \quad \mathbf{(22)}$$

also

$$b = R\left(1 - \sqrt{1 - \frac{2Hw}{s}}\right) \qquad \text{"Long" Formula} \quad \mathbf{(22a)}$$

79. Theoretical and Practical Profiles. In a manner similar to gravity dams, the theoretical profile suitable for an arched dam is a triangle having its apex at the extreme water level, its base width being dependent on the prescribed limiting pressure. Successful examples have proved that a very high value for s, the maximum stress, can be adopted with safety. If it were not for this, the profitable use of arched dams would be restricted within the narrow limits of a short admissible radius, as with a low limit pressure the section would equal that of a gravity dam.

The practical profile is a trapezoid, a narrow crest being necessary. The water pressure area acting on an arched dam, is naturally similar to that in a gravity dam, the difference being, however, that there is no overturning moment when reverse pressure occurs as in a weir. The difference or unbalanced pressure acting at any point is simply the difference of the direct and the reverse forces. The areas of pressure on both sides, therefore, vary with the squares of their respective depths.

The water pressure on an arch acts normally to the surface of its back and is radial in direction; consequently the true line of pressure in the arch ring corresponds with the curvature of the arch and has no tendency to depart from this condition. There is, therefore, no such tendency to rupture as is the case in a horizontal circular arch subjected to vertical rather than radial pressure. This property conduces largely to the stability of an arch under liquid pressure. This condition is not strictly applicable in its entirety to the case of a segment of a circle held rigidly between abutments as the arch is then partly in the position of a beam. The complication of stress involved is, however, too abstruse for practical consideration.

80. Correct Profile. As we have already seen, the correct profile of the arched dam is a triangle modified into a trapezoid with a narrow crest. With regard to arch stresses, the most favorable outline is that with the back of the extrados vertical. The reason for this is that the vertical stress due to the weight of the arch, although it acts on a different plane from the tangential stresses in the arch ring, still has a definable influence on the maximum induced stress in the arch ring. The vertical pressure produces a transverse expansion which may be expressed as $W \times E \times m$, in which E is the coefficient of elasticity of the material and m that of transverse dilation. This tends, when the extrados is vertical, to diminish the maximum stress in the section; whereas when the intrados is vertical and the back inclined, the modification of the distribution of pressure is unfavorable, the maximum stress being augmented. When the trapezoidal profile is equiangular, an intermediate or neutral condition exists. A profile with vertical extrados should, therefore, be adopted whenever practicable.

In very high dams, however, the pressure on the horizontal plane of the base due to the weight of the structure, becomes so great as to even exceed that in the arch ring; consequently it is necessary to adopt an equiangular profile in order to bring the center of pressure at, or near to, the center of the base, so as to reduce the ratio of maximum pressure to average pressure to a minimum. As stated in the previous section, when a vertical through the center of gravity of the profile passes through the center of the base, the maximum pressure equals the average, or $s = s_1$.

81. Support of Vertical Water Loads in Arched Dams. When the back of an arched dam is inclined, the weight of the water over it is supported by the base, the horizontal pressure of the water alone acting on the arch and being conveyed to the abutments. In the case of inclined arch buttress dams, however, a portion of the vertical load is carried by the arch, increasing its thrust above what is due to the horizontal water pressure alone. This is due to overhang, i. e., when the c. g falls outside the base.

82. Crest Width. The crest width of arched dams can be safely made much less than that of gravity dams and a rule of

$$k=\frac{1}{2}\sqrt{H} \tag{23}$$

would seem to answer the purpose, unless reinforcement is used, when it can be made less.

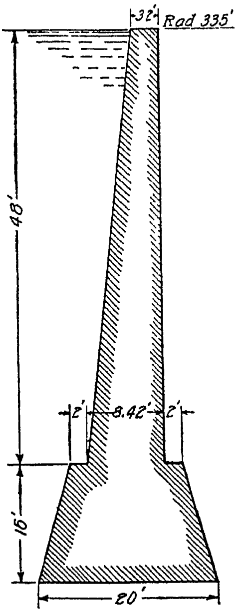


Fig 59 Section of Old Bear Valley Dam

EXAMPLES OF ARCHED DAMS

The following actual examples of arched dams will now be given.

83. Bear Valley Dam. This small work, Fig. 59, is the most remarkable arched dam in existence and forms a valuable example of the enormous theoretical stresses which this type of vertical arch can stand. The mean radius being 335 feet according to formula (21) the unit stress will be

$$\frac{RHw}{b}=60 \text{ tons, nearly}$$

This section would be better if reversed. The actual stress is probably half this amount. This work has now been superseded by a new dam built below it, Fig. 77, section 103.

84. Pathfinder Dam. This immense work, Fig. 60, is built to a radius of 150 feet measured to the center of the crest. That, however, at the extrados of the base of the section is 186 feet and this quantity has to be used for the value of R in the long formula (22). The unit stress then works out to 18 tons, nearly. The actual stress in the lowest arch ring is undoubtedly much less, for the reason

that the base must absorb so large a proportion of the thrust that very little is transmitted to the sides of the canyon. The exact determination of the proportion transmitted in the higher rings is an indeterminate problem, and the only safe method is to assume with regard to tangential arch stress that the arch stands clear of

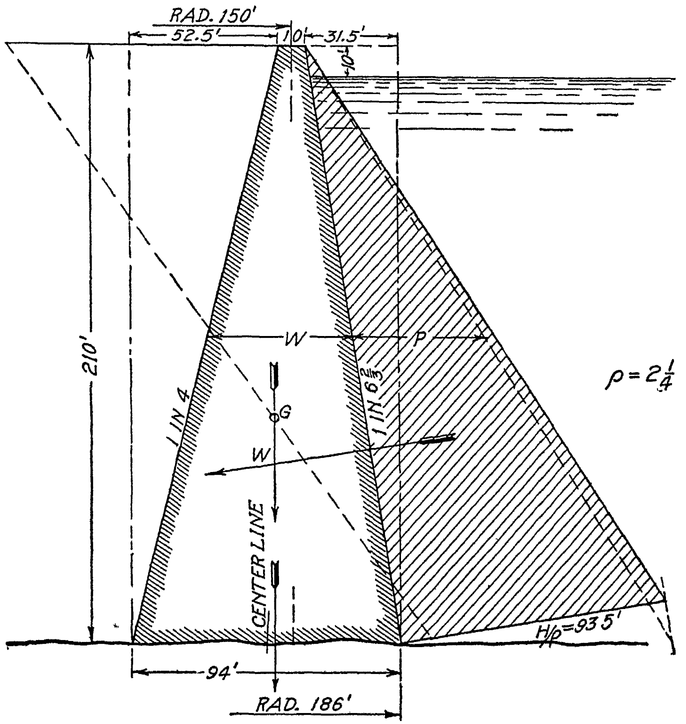


Fig 60. Section of Pathfinder Dam

the base. This will leave a large but indeterminate factor of safety and enable the adoption of a high value for s, the maximum unit stress.

The profile of the dam is nearly equiangular in outline. This is necessary in so high a dam in order to bring the vertical resultants (W) R. E. and (N) R. F. as near the center as possible with the object of bringing the ratio of maximum to mean stress as low as possible.

The estimation of the exact positions of W and of N is made analytically as below.

There are only two areas to be considered, that of the water overlying the inclined back (v) and that of the dam itself (W). Dividing v by $2\frac{1}{4}$ (the assumed specific gravity of the material), reduces it to an equivalent area of concrete or masonry.

$$v=\frac{210\times 31.5}{2\times 2.25}=1470=103 \text{ tons}$$

$$W=\frac{104}{2}\times 210=\underline{10920}=\underline{768} \text{ tons}$$

$$\text{Total, or } N=12390=871 \text{ tons}$$

Using formula (7), Part I, the c.g. of W is 50.8 distant from the toe of the profile, then q or the distance of the incidence of W from the center point of the base is $50.8-\frac{94}{2}=3.8$

The value of s_1, or the mean unit stress is $\frac{W}{b}$, or $\frac{768}{94}=8.1$ tons and $m=1+\frac{6q}{b}=1+\frac{6\times 3.8}{94}=1.24$; then $s=\frac{mW}{b}=1.24\times 8.1=10.1$ tons.

For Reservoir Full, to find the position of N, moments will be taken about the toe as follows

$$\begin{aligned} \text{Moment of } v &= 103\times 83.5= 8600\\ \text{Moment of } W &= 768\times 50.8=\underline{39014}\\ \text{Total } N &= \overline{871} \qquad =\overline{47614} \end{aligned}$$

then $x=\frac{47614}{871}=54.6$, whence $q=54.6-\frac{94}{2}=7.6$ feet and $s_1=\frac{N}{b}=\frac{871}{94}=9.26$. By formula (9), Part I, $m=1+\frac{7.6\times 6}{94}=1.48$.

$$\therefore \qquad s=9.26\times 1.48=13.7 \text{ tons}$$

From this it is evident that the unit stress in the base, due to vertical load only, is a high figure. It could be reduced by still further inclining the back; on the contrary, if the back were vertical N would equal W. Let this latter case be considered. The distance of the c. g. of the profile from the heel will then be by formula (7a), Part I

$$x=\frac{1}{3}\left(b+\frac{a^2}{a+b}\right)=31.66$$

and the value of q will be $\frac{94}{2}-31.66=15.33$ feet

$$s_1 \text{ as before} = \frac{W}{b} = 8.1 \text{ tons}$$

Then

$$m = 1 + \frac{92}{94} = 1.98 \text{ and } s = 8.1 \times 1.98 = 16 \text{ tons}$$

This stress is greater than that of N in the previous working which proves that the forward tilt given to these high dams is necessary to reduce the maximum unit stress on the base to a reasonable limit. A more equiangular profile would give even better results.

85. Shoshone Dam. The Shoshone dam, Fig. 61, is designed on lines identical with the last example. It has the distinction of being the highest dam in the world but has recently lost this

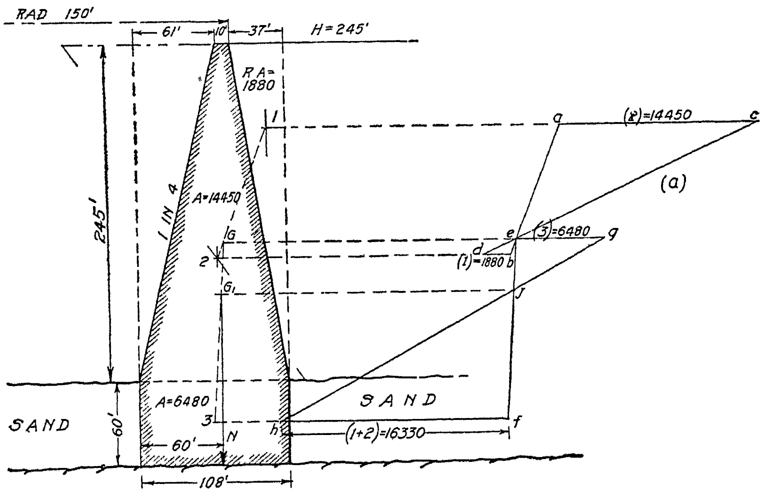


Fig 61 Profile and Force Diagram for Shoshone Dam

preëminence, as the Arrow Rock, quite lately constructed, Fig. 37, Part I, is actually 35 feet higher. This work is also in the United States. The incidents of the resultants Reservoir Empty and Reservoir Full, which will be explained later, have been shown graphically, and the analytical computation is given below. The vertical forces taken from left to right are (1), area 6480; (2), 14,450; (3), water overlying back, reduced area 1880; total 22,810.

Taking moments about the toe of the base, the distance of (1) is 54 feet, of (2) calculated by formula (7), Part I, is 58.3, and of (3) is 95 feet, roughly.

Then $(6480 \times 54) + (14450 \times 58.3) + (1880 \times 95) = 22810 \times x$.

$\therefore \quad x = 60$ feet, nearly.

The value of q for N then is $60 - \frac{108}{2} = 6$ feet.

Now $s_1 = \frac{N}{b} = \frac{22810}{108} = 211$, and by formula (9), Part I, $s = 211 \times \frac{(108+36)}{108} = 281$ square feet $= \frac{281 \times 2.4}{32} = 21$ tons, nearly.

The maximum arch unit stress by formula (22) is as follows: the radius of the extrados of the base being 197 feet the fraction $\frac{b}{R} = \frac{108}{197} = .55$ and $H = 245$ therefore $s = \frac{2HW}{\frac{b}{R} \times \left(2 - \frac{b}{R}\right)} = \frac{2 \times 245 \times 1}{.55 \times 1.45 \times 32} = \frac{490}{25.5} = 19.2$ tons.

Below the level 60 ft. above base, the stress on the arch does not increase. The arch stress is less than that due to vertical pressure N. This base should undoubtedly have been widened, the battered faces being carried down to the base, not cut off by vertical lines at the 60-foot level.

Center of Pressure—New Graphical Method. In order to find the center of pressure in a case like Fig. 61, where the lines of forces (1) and (2) are close together, the ordinary method of using a force and funicular polygon involves crowding of the lines so that accuracy is difficult to attain. Another method now will be explained which is on the same principle as that of the intersection of cross lines used for finding the c. g. of a trapezoid.

In Fig. 61, first the c.g.'s of the three forces are found (1) the water pressure area divided by ρ or 2.4 which equals 1880 square feet, (2) the upper trapezoidal part of the dam area 14,450, and (3) the lower rectangular area 6480. Then (1) is joined to (2) and this line projected on one side in any location as at b in Fig. 61a.

From a, ac is set off horizontally equal to (2) or 14,450 and from b, bd is drawn equal to (1) or 1880; cd is then drawn and its intersection with ab at e gives the position of the resultant 1-2, which can now be projected on the profile at G. To obtain the resultant of the components (1-2) with (3) the line G-3 is drawn on the profile

and a parallel to it drawn from e on Fig. 61a, intersecting the horizontal through (3) at f. From e, eg is laid off horizontally equal to (3) or 6480 and from f, fh equal to (1+2) or 1880+14,450=16,330. hg is then drawn and its intersection with ef at j is the centroid of the three forces, which projected on the profile to G_1 on the line G-3 gives the location of the vertical resultant of 1+2+3.

86. Sweetwater Dam. The profile of the Sweetwater dam in California is given in Fig. 62. The original crest of the dam

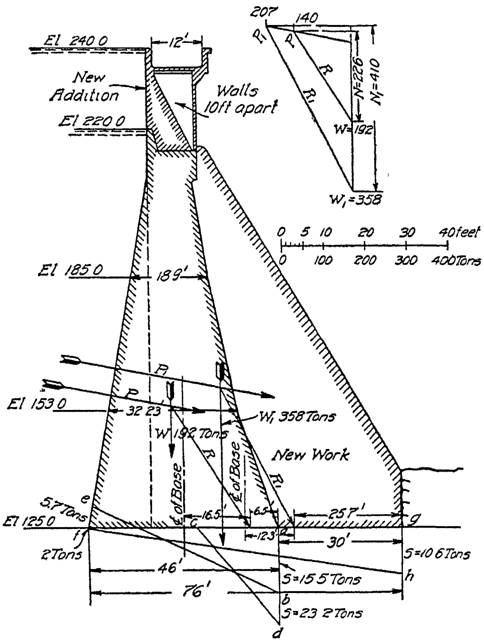


Fig 62 Graphical Analysis of Sweetwater Dam, California

was at *El.* 220, or 95 feet above the base. Under these conditions the dam depended for its stability on its arched plan. If considered as a gravity dam with allowable tension at the heel, the vertical pressure area is the triangle abe, here $q=16.5$ and m works out to 3.15. $N=226$ tons and $b=46$ feet whence $s=\frac{mN}{b}=\frac{3.15\times 226}{46}=15.5$ tons which is set down from a to b.

The tension at the heel $=s_2-\frac{2N}{b}=15.5-9.82=5.7$ tons

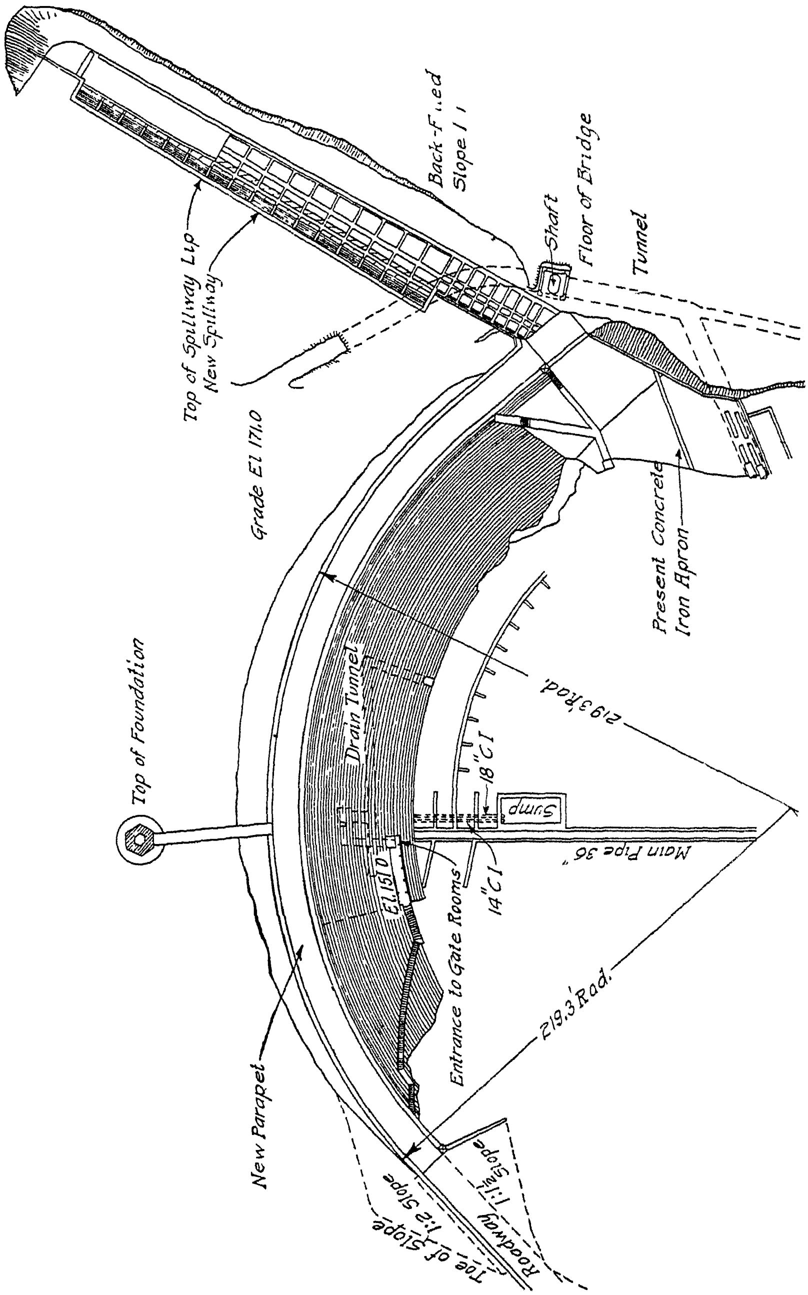


Fig 63. Location Plan of Sweetwater Dam, Showing Alterations

Value of S When Heel Is Unable to Take Tention. If the heel is unable to take tension, the pressure triangle will then be *adc* in which $ac = 3$ times the distance of the incidence of R from the toe, or $3 \times 6.5 = 19.5$ feet and s is obtained by the following formula

$$s = \frac{4}{3} \times \frac{N \text{ or } W}{b - 2q} \qquad \textbf{(24)}$$

here $s = \dfrac{4 \times 226}{3 \times 46 - 33} = 23.2$ tons

This dam has lately been raised to *El* 240, or by 20, feet and by the addition of a mass of concrete at the rear transformed into a gravity dam. The resultant due to this addition is R_1 on the diagram. s works out to 10.6 tons and there is no tension at the heel. Any bond between the new wall and the old has been studiously avoided. The new work is reinforced with cross bars and the rear mass tied into the superstructure. Fig. 63 is a plan of the dam as altered.

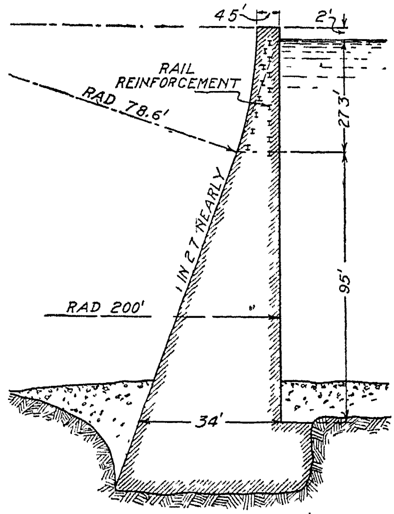


Fig 64 Profile of Barossa Dam

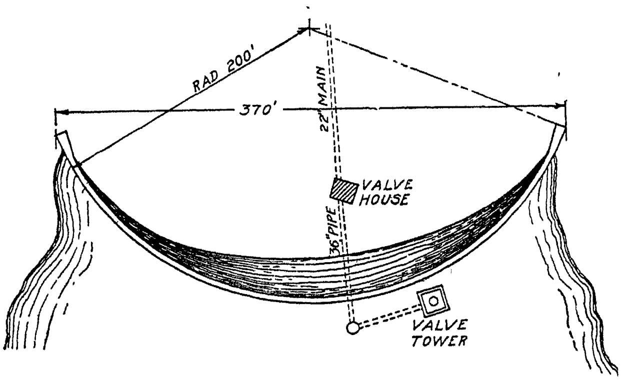


Fig 65 Site Plan of Barossa Dam

87. Barossa Dam. This dam, Fig. 64, is an Australian work, and although of quite moderate dimensions is a model of good and bold design.

The back is vertical and the fore batter is nearly 1 in 2.7. The outline is not trapezoidal but pentagonal, viz, a square crest imposed on a triangle, the face joined with the hypothenuse of the latter by a curve. The crest is slender, being only $4\frac{1}{2}$ feet wide, but is strengthened by rows of 40-pound iron rails, fished together, built into the concrete. The maximum arch stress works out to $17\frac{1}{4}$ tons, the corresponding vertical stress on base to $6\frac{3}{4}$ tons. Fig. 65 is a site plan of the work.

88. Lithgow Dam. Another example very similar to the last is the Lithgow dam, No. 2, Fig. 66. The arch stress in this works out by the short formula to nearly 13 tons; the radius is only 100 feet, the vertical stress works out to 7 tons.

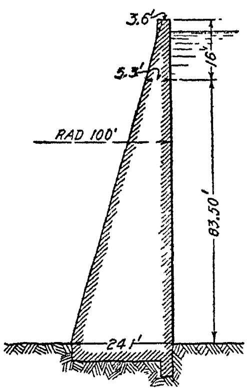


Fig 66 Profile of Lithgow Dam

Arched dams abut either on the solid rocky banks of a canyon or else on the end of a gravity dam. In cases where a narrow deep central channel occurs in a river, this portion can advantageously be closed by an arched dam, while the flanks on which the arch abuts can be gravity dams aligned tangential to the arch at each end. The dam will thus consist of a central arch with two inclined straight continuations. The plan of the Roosevelt dam, Fig. 26, Part I, will give an idea of this class of work.

89. Burrin Juick Subsidiary Dam. In Fig. 67 is shown the profile of a temporary reinforced arched dam for domestic water supply at Barren Jack, or Burrin Juick, Australia. The reinforcement consists of iron rails. The unit arch pressure at the base works out to 21 tons, nearly. Reinforcement of permanent dams down to the base is not desirable, as the metal may corrode in time and cause failure, although the possibility is often stoutly denied. The main Burrin Juick dam is given in Part I, Fig. 36.

90. Dams with Variable Radii. The use of dams of the type just described, is generally confined, as previously noted, to narrow gorges with steep sloping sides in which the length of the dam at the level of the bed of the canyon is but a small proportion of that at the crest. The radius of curvature is usually fixed with regard to the

length of chord at the latter level, consequently at the deepest level, the curvature will be so slight that arch action will be absent and the lower part of the dam will be subject to beam stresses, i.e., to tension as well as compression. In order to obviate this, in some recent examples the radius of curvature at the base is made less than that at the crest, and all the way up, the angle subtending the chord of the arc, which is variable in length retains the same measure throughout. This involves a change in the radius corresponding to the variable span of the arch. The further advantage is obtained, of reduction in the unit stress in the arch ring and in rendering the stress more uniform throughout. In very high dams, however, the base width cannot be much reduced as otherwise the limit stress due to the vertical loading will be exceeded. This arrangement of varying radii is somewhat similar to that used in the differential multiple arch given later.

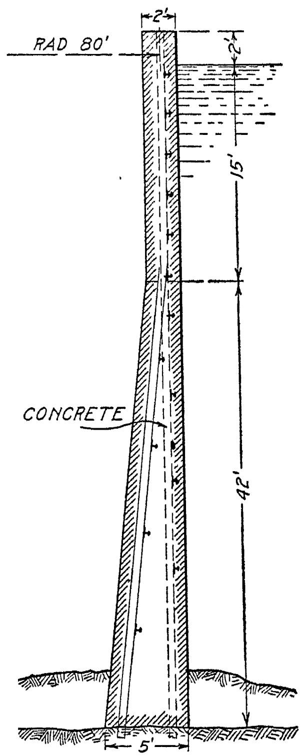


Fig 67 Profile of Burrin Juick Subsidiary Dam

MULTIPLE ARCH OR HOLLOW ARCH BUTTRESS DAMS

91. Multiple Arch Generally More Useful Than Single Arch Dams. It is evident that a dam which consists of a single vertical arch is suitable only for a narrow gorge with rock sides on which the arch can abut, as well as a rock bed; consequently its use is strictly limited to sites where such conditions are obtainable. A rock foundation is also essential for gravity dams, the unit compression on the base of which is too high for any material other than rock.

The advantages inherent in the vertical arch, which are considerable, can however be retained by use of the so-termed multiple or scallop arched dam. This consists of a series of vertical or inclined arches, semicircular or segmental on plan, the thrust of

which is carried by buttresses. The arrangement is, in fact, identical with that of a masonry arched bridge. If the latter be considered as turned over on its side, the piers will represent the buttresses. In the case of a wide river crossing, with a bed of clay, boulders, or sand, the hollow buttressed and slab buttressed dams are the only ones that can well be employed with safety. The wide spread that can be given to the base of the structure in these two types enables the unit pressure on the base to be brought as low as from 2 to 4 tons per square foot.

As has already been noticed in section 78, the arch is peculiarly well suited for economical construction. This is due to the fact that the liquid pressure to which the arch is subjected is normal to the surface and radial in direction. The pressure lines in the interior of the arch ring correspond with its curvature and consequently the arch can only be in compression; thus steel reinforcement is unnecessary except in a small degree near the crest in order to care for temperature stresses. In slab dams, on the other hand, the deck is composed of flat slabs which have to be heavily reinforced. The spacing of the buttresses for slabs is limited to 15 to 20 feet, whereas in hollow arch dams there is practically no limit to the spans which may be adopted. Another point is, that the extreme compressive fiber stress on the concrete in deck slabs is limited to five hundred to six hundred and fifty pounds per square inch; in an arch, on the other hand, the whole section is in compression which is thereby spread over a much greater area. For the reasons above given the arch type now under consideration should be a cheaper and more scientific construction than the slab type in spite of the higher cost of forms.

92. Mir Alam Dam. The first example given is that of the Mir Alam tank dam, Fig. 68. This remarkable pioneer structure was built about the year 1806, by a French engineer in the service of H. H. the Nizam of Hyderabad in Southern India. The alignment of the dam is on a wide curve and it consists of a series of vertical semicircular arches of various spans which abut on short buttress piers, Fig. 69. The spans vary from 83 to 138 feet, the one in Fig. 68 being of 122 feet. The maximum height is 33 feet. Water has been known to overtop the crest. The length of the dam is over 3000 feet.

On account of the inequality of the spans, the adoption of the semicircular form of arch is evidently a most judicious measure, for the reason that an arch of this form under liquid pressure exerts no lateral thrust at the springing. The water pressure being radial in direction, cross pressure in the half arches in the line of the springing is balanced and in equilibrium. Whatever thrust is exerted is not in the direction of the axis of the dam but that of the buttress piers. On the other hand, if the arches were segmental in outline the terminal thrust is intermediate between the two axes, and when resolved in two directions one component acts along the axis of

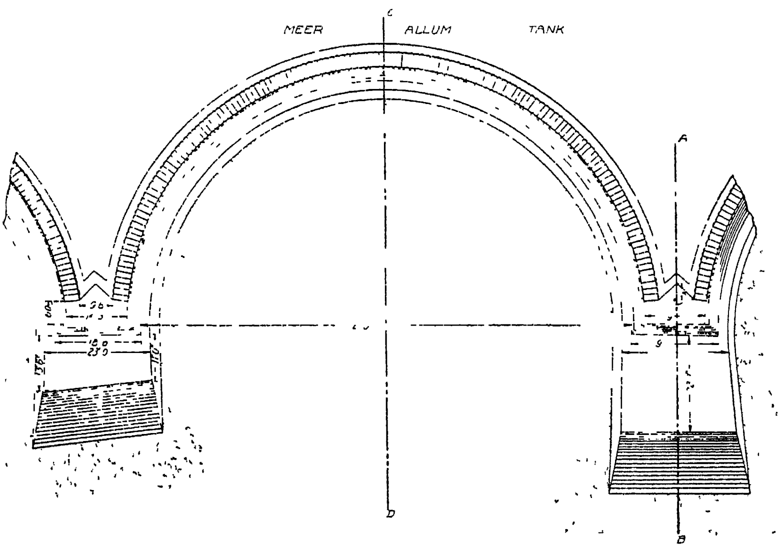


Fig 68 Plan of One Arch of Mir Alam Dam
This remarkable pioneer dam was built in 1806, and consisted of 21 such arches.

the dam. This has to be met, either by the abutment, if it is an end span, or else by the corresponding thrust of the adjoining half arch. The other component is carried by the buttress; therefore, if segmental arches are used, the spans should be equal in order to avoid inequality of thrust. Longer buttresses will also be requisite. The whole of this work is built of coursed rubble masonry in lime mortar; the unit stress in the arch ring at the base, using the short formula (21), $\frac{(RHw)}{b}$ works out to $\frac{68\times33\times1}{14\times32}=5$ tons, nearly. The dam, therefore, forms an economical design.

The buttress piers are shown in section in Fig. 70, the section being taken through AB of Fig. 68. In this work the buttress piers are very short, projecting only 25 feet beyond the spring line of the arches, and being altogether only 35 feet long. This length and the corresponding weight would clearly be inadequate to with-

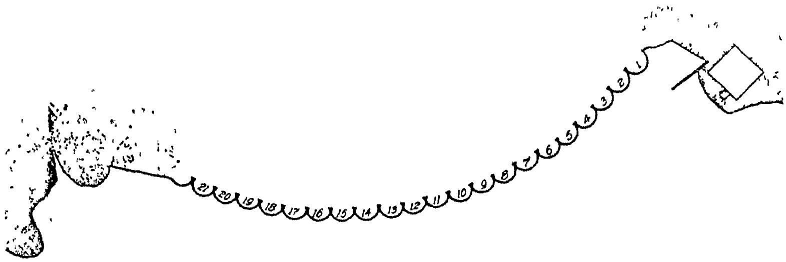


Fig. 69 Plan of Entire Mir Alam Dam

stand the immense horizontal thrust which is equivalent to $\frac{H^2}{2}lw = \frac{33^2 \times 1 \times 146}{2 \times 32} = 2500$ tons, nearly.

It is evident that if the buttress pier slides or overturns, the arches behind it must follow, for which reason the two half arches and the buttress pier cannot be considered as separate entities but as actually forming one whole, and consequently the effective length of the base must extend from the toe of the buttress right back to the extrados of the two adjoining arches. At, or a little in the

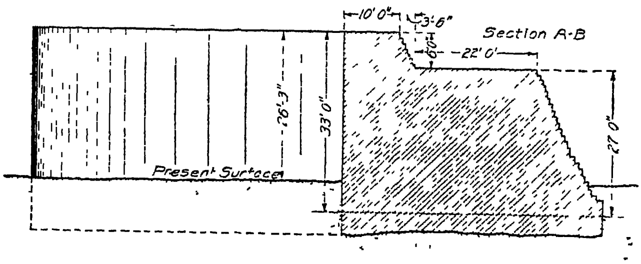


Fig. 70. Section of Buttress Pier of Mir Alam Dam Taken through AB of Fig. 68

rear of the spring line, the base is split up into two forked curved continuations. The weight of these arms, i.e., of the adjoining half arches, has consequently to be included with that of the buttress proper when the stability of the structure against overturning or sliding is estimated.

93. Stresses in Buttress. In the transverse section, Fig. 71, taken through CD of Fig. 68, the graphical calculations establish the fact that the resultant line R intersects the base, thus lengthened, at a point short of its center; the direction of the resultant R is also satisfactory as regards the angle of frictional resistance.

R_1 is the resultant on the supposition that the buttress is nonexistent. Its incidence on the base proves that the arch is stable without the buttress, which is therefore actually superfluous. With regard to sliding on the base, $P = 2500$ and $W = 6828$ tons.

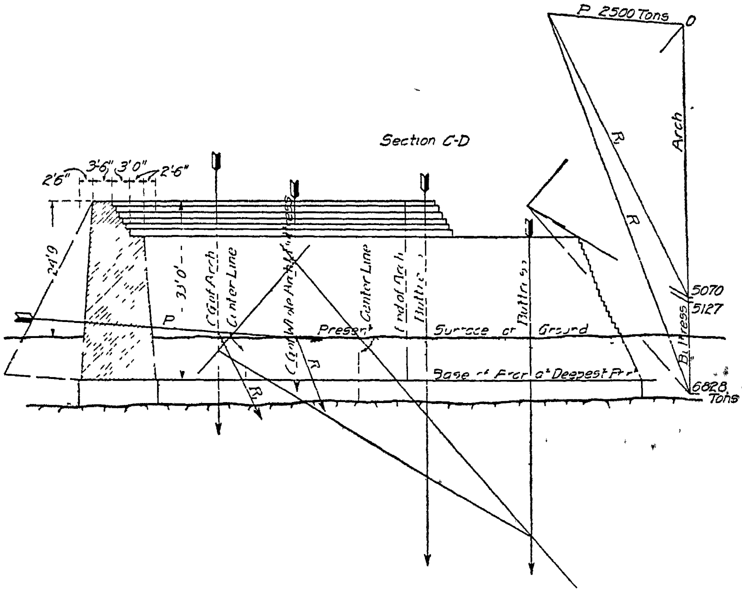


Fig 71. Transverse Section of Mir Alam Dam Taken through CD, Fig. 68

The coefficient of friction being .7 the factor of safety against sliding is nearly 2. If the arch were altered on plan from a semicircle to a segment of a circle, the radius would of necessity be increased, and the stress with it; a thicker arch would, therefore, be required. This would not quite compensate for the reduced length of arch, but on the other hand, owing to the crown being depressed, the effective base width would be reduced and would have to be made good by lengthening the buttress piers. What particular disposition of arch and buttress would be the most economical is a

matter which could only be worked out by means of a number of trial designs. The ratio of versed sine to span should vary from $\frac{1}{4}$ to $\frac{1}{2}$. Arcs subtending from 135 to 120 degrees are stated to be the most economical in material.

94. Belubula Dam. There are not as yet very many modern examples of arch buttress dams, but each year increases their num-

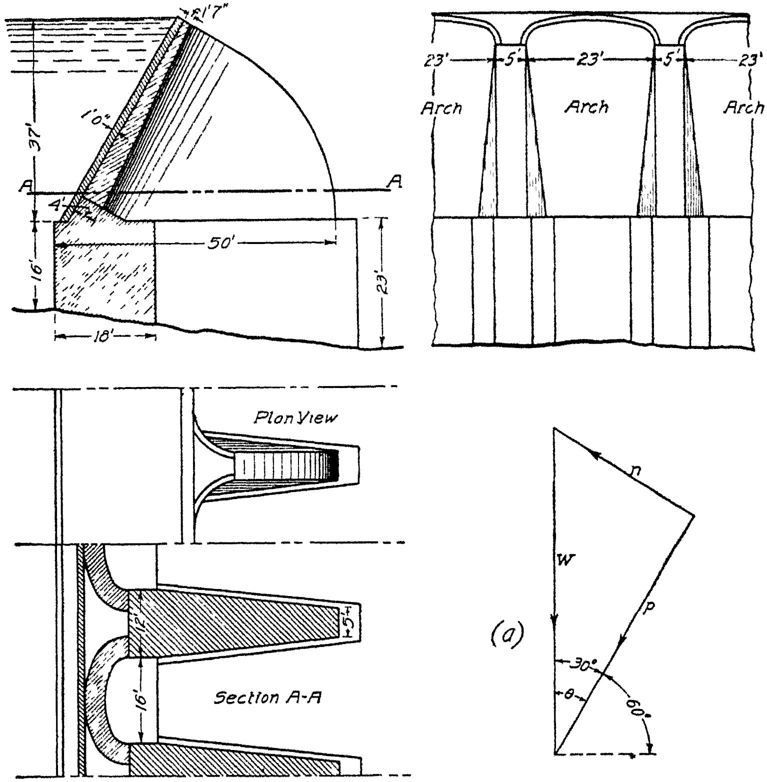


Fig 72 Profile Sections and Force Diagram for Belubula Dam, New South Wales

ber. The Mir Alam dam has remained resting on its laurels without a rival for over 100 years, but the time has come when this type is being largely adopted. Fig. 72 shows an early example of a segmental panel arch dam. It is the Belubula dam in New South Wales. The arch crest is 37 feet above the base, very nearly the same as in the last example. The arches, which are inclined 60 degrees to the horizontal are built on a high solid platform which obliterates

inequalities in the rock foundation. This platform is 16 to 23 feet high, so that the total height of the dam is over 50 feet. The spans are 16 feet, with buttresses 12 feet wide at the spring line, tapering to a thickness of 5 feet at the toe; they are 40 feet long. The buttress piers, which form quadrants of a circle in elevation, diminish in thickness by steps from the base up, these insets corresponding with similar ones in the arch itself. These steps are not shown in the drawing; the arch also is drawn as if in one straight batter. The arches are elliptical in form, and the spandrels are filled up flush with the crown, presenting a flat surface toward the water.

Some of the features of this design are open to objection: *First*, the filling in of the arch spandrels entirely abrogates the advantage accruing to arches under liquid pressure. The direction of the water pressure in this case is not radial but normal to the rear slope, thus exactly reproducing the statical condition of a horizontal arch bridge. The pressure, therefore, increases from the crown to the haunches and is parabolic, not circular, in curvature. The arches should have been circular, not elliptical, and the spandrels left empty to allow of a radial pressure which partly balances itself. *Second*, the stepping in of the intrados of the arch complicates the construction. A plain batter would be easier to build, particularly in concrete. *Third*, the tapering of the buttress piers toward the toe is quite indefensible; the stress does not decrease but with the center of pressure at the center of the base as in this case, the stress will be uniform throughout.

95. Inclination of Arch to Vertical. The inclination of the axis of the arch to the vertical is generally a desirable, in fact, a necessary feature when segmental arched panels are used; the weight of water carried is of value in depressing the final resultant line to a suitable angle for resistance to shearing stress. As noted in section 90, the weight of the water overlying the arch does not increase the unit stress in the arch ring. Consequently, any inclination of axis can be adopted without in any way increasing the unit stresses due to the water pressure.

When an arch is vertical it is clear that the water pressure is all conveyed to the abutments and the weight of the arch to its base. When an arch lies horizontally under water pressure both the weight of the water and that of the arch itself are conveyed

to the abutment; when in an intermediate position part of the weight of the arch is carried to the base and part to the abutments.

With regard to water pressure, the thrust being normal to the extrados of the arch the whole is carried by the abutments. In the case of arches which do not overreach their base the weight of water overlying the inclined back is conveyed to the base. In any case the unit stress in the arch $\frac{(RHw)}{b}$ cannot exceed that due to horizontal thrust. The total water pressure is greater with an inclined back, as the length of surface acted on is increased. In the diagram, Fig. 72a, the vertical load line W represents the weight of one unit or one cubic foot of the arch ring which is equal to $w\rho$. This force is resolved in two directions, one p, parallel to the axis of the arch, and the other n, normal to the former. The force $n=W \sin \theta$, θ being the inclination of the arch axis to the vertical and $p=W \cos \theta$. The unit stress developed by the radial force n is similar to that produced by the water pressure which is also radial in direction and is R_1n; but R_1, the radius in this case, is the mean radius, the pressure being internal, not external. The unit stress s_1 will then be

$$s_1=R_1w\rho \sin \theta \tag{25}$$

When θ is 30° $\sin \theta=\frac{1}{2}$; when 45°, $\sin \theta=\frac{2}{3}$.

It will easily be understood that this unit stress due to n does not accumulate, but is the same at the first foot depth of the arch as it is at the bottom; the width of the lamina also does not affect it. However, the component p does accumulate, and the expression $w\rho \cos \theta$ should be multiplied by the inclined height H_1, lying above the base under consideration. As $H_1=H \sec \theta$, the unit compressive stress at the base will be $\frac{Hw\rho b_1}{b}$, in which b_1 is the mean width of the arch. If the arch were a rectangle, not a trapezoid, s would equal $Hw\rho$ simply.

96. Ogden Dam. The Ogden dam, the profile and sectional details of which are shown in Fig. 73, is a notable example of the arch and buttress type. Its height is 100 feet. The inclination of the arches is less than ½ to 1, or about 25 degrees to the vertical.

The profile of the buttress is equinangular except for a small outthrow of the toe. On the whole it must be pronounced a good design, but could be improved in several particulars. For example, the arch is unnecessarily thick at the crest, and could well be reduced from 6 to 2 feet, thus effecting considerable economy. The designers were evidently afraid of the concrete in the arch leaking, and so overlaid the extrados with steel plates. A greater thickness of arch causing it to possess less liability to percolation under pressure, could have been provided by increasing the span and radius of the

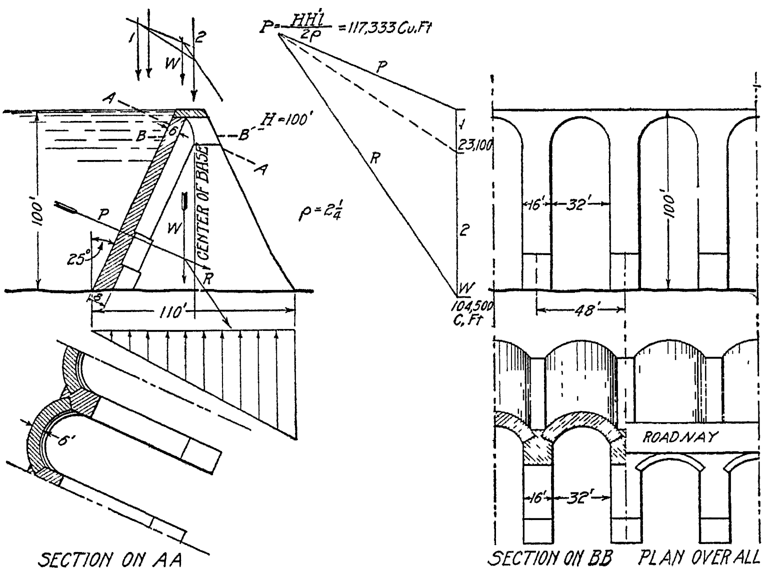


Fig. 73. Profile and Sections of Ogden Dam

arches. The design consequently would be improved by adopting larger spans, say 100 feet; buttresses, say, 25 feet thick, their length being dependent on the width of base required to provide sufficient moment of resistance; and further, the inclination of the arches might require increasing to bring the center of pressure at, or close to the center of the buttress. The finish of the crest by another arch forming a roadway is an excellent arrangement, and is well suited for a bulkhead dam; for an overfall, on the other hand, the curved crest is preferable on account of the increased length of overflow provided. The stress diagram shows that the value of the

vertical load N is 155,000 cubic feet or 10,598 tons, ρ being taken at $2\frac{1}{4}$. The incidence of R on the base, is 5 feet from the center, whence $q=5$, and by formula (9), Part I

$$s=\frac{N}{A}\times\left(1+\frac{6q}{b}\right)=\frac{10598}{110\times16}\times\frac{140}{110}=8.91 \text{ tons}$$

the dimensions of A, the area of the base, being 110×16 feet. The pressure on the arch ring at the base by the short formula works out to $\frac{24\times100}{8\times32}=9.4$ tons.

The contents of the dam per foot run amounts to $\frac{104,500}{48}=$ 2,177 cubic feet; that of a gravity dam would be about 3,500 cubic feet per foot run, making a saving in favor of the arched type of nearly 30 per cent. With a better disposition of the parts as indicated above, the saving would be increased to 40 or 50 per cent. Actually the saving amounted to only 12 per cent; this was owing to the steel covering which, as we have seen, could have been dispensed with.

97. Design for Multiple Arch Dam. Fig. 74 is a design for a segmental arch panel dam, or rather, weir. The height of the crest is 64 feet above base with 5 feet of water passing over; the apex of the triangle of water pressure will then be 69 feet above the base. The inclination given the axis, which is coincident with that of the spring line and the intrados, is 60 degrees with the horizon.

In designing such a work, the following salient points first require consideration.

(1) *Width of Span.* This, it is deemed for economical reasons should be not less than the height of crest unless the state of the foundation requires a low unit stress. In the Mir Alam dam the span is over four times the depth of water upheld. In the present case it will be made the same, that is, 64 feet.

(2) *Thickness of Buttress Piers.* As with bridge piers, the width should be at least sufficient to accommodate the skew-backs of the two arches; a width of 12 feet or about $\frac{1}{5}$ span will effect this.

(3) *Radius and Versed Sine.* The radius will be made 40 feet; this allows a versed sine of $\frac{1}{4}$ span, or 16 feet, which is considered to be about the flattest proportion to afford a good curva-

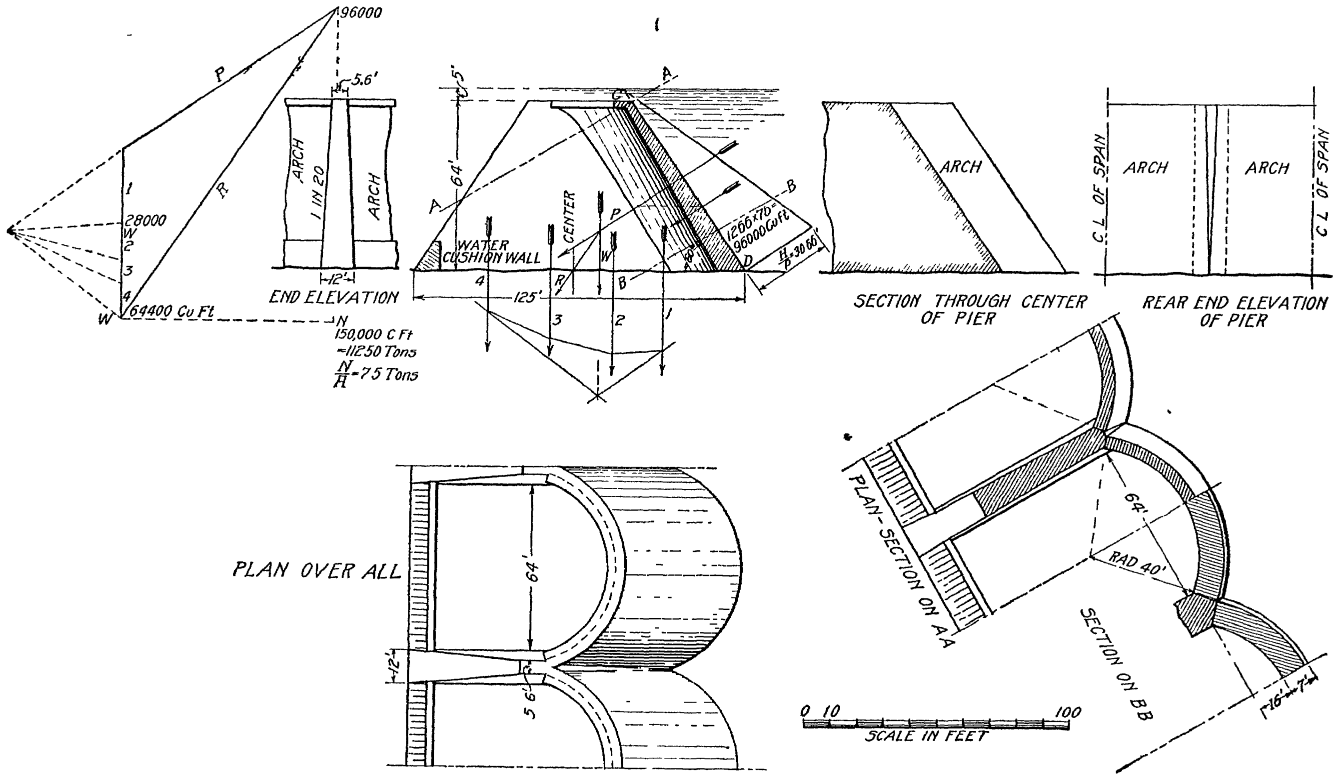


Fig 74. Profile Plans and Sections of Segmental Arch Panel Dam

ture, the greater the length of the arc, the more its condition will approximate to that of a circular arch, under liquid pressure.

(4) *Thickness of Arch.* This must first be assumed, as its thickness depends on R, the radius of the extrados, as well as on the value assigned to s_1, the limiting pressure. This latter will be fixed at below 15 tons, a value by no means excessive for arches under liquid pressure. With a base width of 7 feet, the radius of the extrados will be 47 feet. The base will be considered, not at the extreme depth of 64 feet below crest, but at the point marked D, where a line normal to the base of the inclined intrados cuts the extrados of the arch. H will, therefore, be 60 feet, allowing for the reverse pressure. The stress due to the water pressure, using the short formula (**21**), section 78, will be

$$s_1=\frac{RHw}{b}=\frac{47\times60\times1}{7\times32}=12.6 \text{ tons}$$

To this must be added that due to the weight of the arch ring from formula (**25**), $s_1 = R_1 w\rho \sin\theta$ (the angle θ being 30° and its sine $= \frac{1}{2}$), which in figures will be $\frac{43.5\times3}{2\times40}=1.6$ tons, the total stress being a trifle over 14 tons. The 7-foot base width will then be adopted. ρ is taken as 2.4 and $w\rho=\frac{3}{40}$ ton. The depth of water producing this pressure is taken as 60, not as 65, feet which is $(H+d)$, the reason being that the reverse pressure due to the tail water, which must be at least level with the water cushion bar wall, will reduce the effective depth to 60 feet, during flood conditions.

98. Reverse Water Pressure. The influence of the reverse pressure of water is much more considerable when hydrostatic pressure alone is exerted than is the case with overturning moment. In the case of an upright arch acting as an overfall weir the pressure of the tail water effects a reduction of the pressure to the extent of its area. Thus if A be the area of the upstream water pressure, and a that of the downstream, or tail water, the unbalanced pressure will be their difference, or $A-a$, and will vary as the square of their respective depths. When overturning moment is concerned, the areas have to be multiplied by a third of their depths to represent the moment on the base. The difference of the two will be in that case as the cubes of their respective depths.

99. Crest Width of Arch. The crest width of the arch, according to formula (**23**), should be $\frac{1}{2}\sqrt{H}=3\frac{3}{4}$ feet, nearly. It will be made 3 feet, with a stiffening rib or rim of 3 feet in width. The crest width could be made proportional to the base width, say $.3b$, and if this falls below 2 feet, reinforcement will be required.

The length of the pier base is measured from the extrados of the arch, the two half arches forming, as already explained in section 92, a forked continuation of the buttress pier base.

The battering of the sides of the pier would clearly be a correct procedure, as the pressure diminishes from the base upward. A combined batter of 1 in 10 is adopted, which leaves a crest width of 5.6 feet. The length of the pier base, as also its outline, were determined by trial graphical processes, with the object of maneuvering the center of pressure as near that of the base as possible, so as to equalize the maximum and the mean unit stress as much as possible. This has been effected, as shown by the incidence of the final resultant on the elevation of the buttress pier.

100. Pressure on Foundations. The total imposed weight is measured by N in the force diagram, and is equivalent to 150,000 cubic feet of masonry, which at a specific gravity of 2.4 is equal to $\frac{150{,}000\times 3}{40}=11{,}250$ tons. The average pressure is this quantity divided by the area of the base, or by $125\times 12=1500$ square feet, the quotient being $7\frac{1}{2}$ tons, nearly. The maximum pressure will be the same owing to the incidence of R at the center of the base. This $7\frac{1}{2}$ tons is a very moderate pressure for a hard foundation; if excessive, additional spread should be provided or else the spans reduced. It will be noticed that N greatly exceeds W. This is due to the added weight of water represented by the inclination given to the force line P, which represents the water pressure.

Economy of Multiple Arches. The cubic contents per foot run work out to $\frac{64{,}000}{76}=850$ cubic feet, nearly, the denominator in the fraction being the distance apart of the centers of the buttress piers.

The contents of a gravity weir with base width $\frac{2}{3}(H+d)$ and top $\sqrt{H+d}$, works out to 1,728 cubic feet; the saving in material is therefore over 50 per cent.

101. Differential Arches. Fig. 75 is a study of a differential buttress arch weir. The principle of the differential arch consists in the radius increasing with the height of the arch, the unit stress is thus kept more uniform, and the stress area corresponds more closely with the trapezoidal profile that has necessarily to be adopted, than is the case when a uniform radius is adopted as in Fig. 74.

The arches are supposed to stand on a concrete or masonry platform ten feet high above the deepest part of the river bed, so that sluices if required could be provided below *L.W.L.* which is identical with the floor or fore apron level. The height is 35 feet to crest level. The depth of film passing over the crest is assumed at 5 feet and the reciprocal depth of tail water is 12 feet. Graphical analyses will be made at two stages, first, when water is at crest level and the river channel below is empty, second, at full flood. The inclination given to the intrados of the arch is 3 vertical to 2 horizontal. The buttresses are placed 31 feet centers, allowing a span of 25 feet at base, here they are 6 feet wide, tapering to 2 feet at crest. The span of the arch thus gradually widens from 25 to 29 feet. The versed sine of the arc is made 5 feet at base and $2\frac{1}{2}$ feet at crest. The radii at these positions are therefore 18.1 and 43.3, respectively, measured to the intrados of the arch. These radii are horizontal, not normal to the intrados as in Fig. 73, and thus vary right through from 18.1 to 43.3 corresponding to the altered versed sine which decreases from 5 to $2\frac{1}{2}$ feet, that half way up being 22 feet.

The thickness of the arch at base is made 2 feet.

Arch Unit Stress. Taking the base radius as 18.1, the base unit stress due to water pressure will be by formula (**21**), $s=\frac{RHw}{b}$; adding that due to the transmitted weight of the arch, formula (**25**), $s=Rw\left(\frac{H}{b}+\rho\sin\theta\right)$, $\sin\theta$ being .6, the expression becomes $18.1w\left[\frac{35}{2}+(2.4\times.6)\right]$, ρ being taken at 2.4, w at $\frac{1}{32}$ ton, whence $s=10.4$ tons, a moderate stress for a vertical arch.

The real thickness of the arch is more like $2\frac{1}{2}$ feet than 2 feet as properly it should be measured horizontally, not normally.

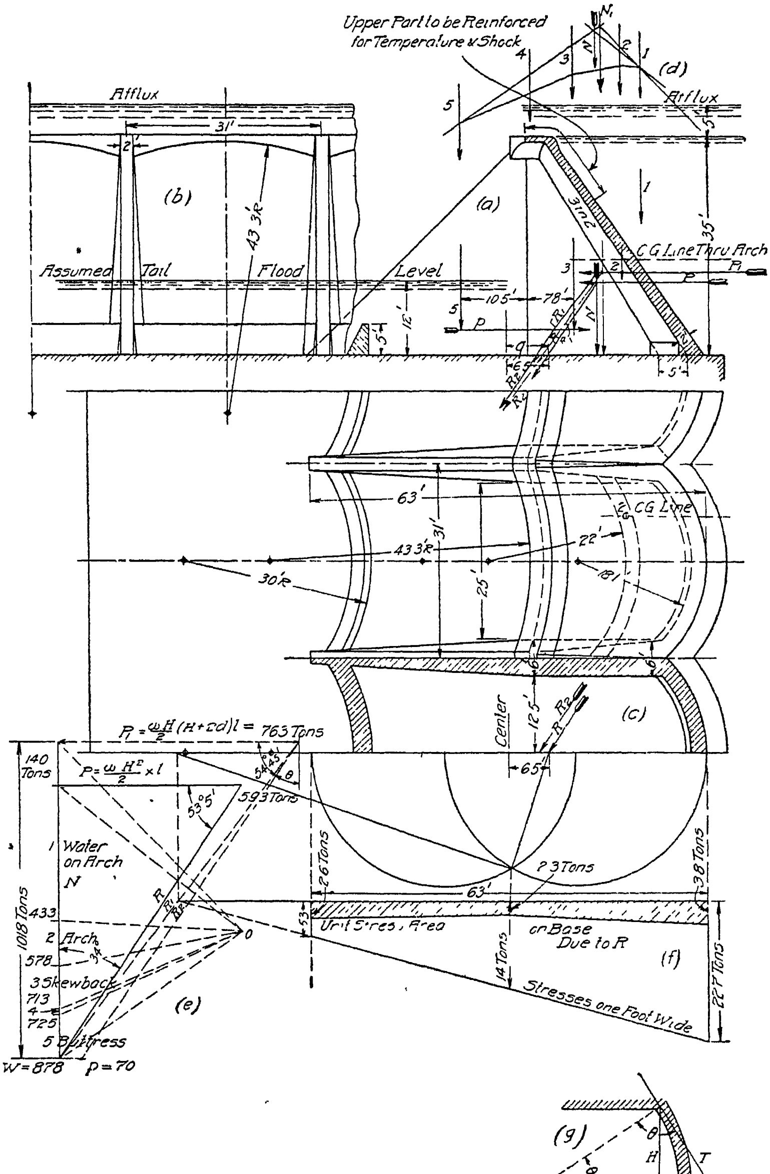


Fig 75. Design Diagrams for Differential Buttress Arch Weir

Load Line. In the force polygon the load line is made up of five weights: (1) that of the overlying water has a content of $\frac{33\times24\times35}{2}=13860$ cubic feet, equivalent to 433 tons; (2) the arch 145 tons; (3) the contents of the pier underlying the arch is found by taking the contents of the whole as if the sides were vertical and deducting the pyramid formed by the side batters. Thus the contents of the whole is $\frac{22\times6\times35}{2}=2310$, that of the pyramid is $\frac{22\times2\times35}{3}=513$, difference 1797, or 135 tons; (4) the weight of the horizontal arch of the crest of the weir, 12 tons; (5) the contents of the buttress, by the prismoidal formula comes to $\frac{1}{6}\,l\,(A_1+4A_m+A_2)$ in which A_1 and A_2 are areas of the ends and A_m of the middle section. Here $A_1=0$, $A_m=\frac{35}{2}\times\frac{6}{2}=52.5$, and $A_2=35\times4=140$; therefore (5) $=\frac{1}{6}\times35\times[0+(4\times52.5)+140]=2042$ cubic feet equivalent to 153 tons.

The total load foots up to 878 tons.

P the horizontal water pressure $=w\frac{H^2}{2}\times l=\frac{1}{32}\times\frac{(35)^2}{2}\times31=593$ tons.

The position of the several vertical forces is obtained as follows: That of 1, a triangular curved prism is at $\frac{1}{3}$ its horizontal width; of 2 is found by formula (7), Part I, and by projection of this level on to the plan. The position of 3 has to be calculated by moments as below.

The lever arm of the whole mass including the battered sides is at $\frac{1}{3}$ width from the vertical end of $7\frac{1}{3}$ feet while that of the pyramidal batter is at $\frac{1}{4}$ the same distance, or $5\frac{1}{2}$ feet.

The statement is then

$$2310\times7\tfrac{1}{3}=(1797\times x)+(514\times5.5)\ \text{ whence } x=7.84 \text{ feet}$$

The position of 5, the battered sloping buttress is obtained by taking the center part 2 feet wide and the outer side batters separately. The c.g. of the former is at $\frac{1}{3}$ the length, $\frac{35}{3}=11\frac{2}{3}$ from the

vertical end, and its contents are $\frac{35^2}{2} \times 2 = 1225$ cubic feet $= 92$ tons. The weight of the whole is 153 tons, so that the side batters will weigh $153 - 92 = 61$ tons, and be $\frac{35}{4} = 8.75$ feet distant from the end. Taking moments about the vertical end, we have

$$153x = (92 \times 11.67) + (61 \times 8.75)$$

$$x = \frac{1606}{153} = 10.5 \text{ feet}$$

Therefore, the incidence of the resultant on the base line measured 6.5 feet upstream from the center point.

In Fig. 75a, $N = 878$ tons and $\frac{N}{b} = 14$; b being 63 feet and $q = 6.5$ feet, whence $m = 1.62$ and the stress on the buttress, if only 1 foot wide, $= 14 \times 1.62 = 22.7$ tons. The compression at the toe $= \frac{2N}{b} - s = (28 - 22.7) = 5.3$ tons. These quantites have now to be divided by the base widths to obtain the unit stresses, which are as follows: at heel, $\frac{22.7}{6} = 3.8$; at center, $\frac{14}{6} = 2.3$; at toe, $\frac{5.3}{2} = 2.6$ tons. This stress area is shown hatched in Fig. 75f.

This stress diagram is useful as showing that owing to the incidence of R being behind the center point the total stress diminishes toward the toe of the buttress, consequently it should be tapered on plan, as has been done. In Fig. 74 it has been shown that the stress being uniform by reason of the incidence of R at the center point of the base, the buttress has been made rectangular in plan at its base. The indicated unit stresses are very light which is a great advantage on a bad foundation.

102. Flood Pressures. The second, or flood stage, will now be investigated. Here the vertical load line N in Fig. 75e is increased by 140 tons, the additional weight of water carried by the arch. The horizontal water pressure P_1 is now 763 tons and $N = 1018$, their resultant being R_1. The reverse pressure due to a depth of 12 feet of water is 70 tons, this combined with R_1, in Fig. 75a, results in R_2 the final resultant. The value of θ is 35° 15′ which is satisfactory. As q scales 5 feet, the unit stresses work out as follows:

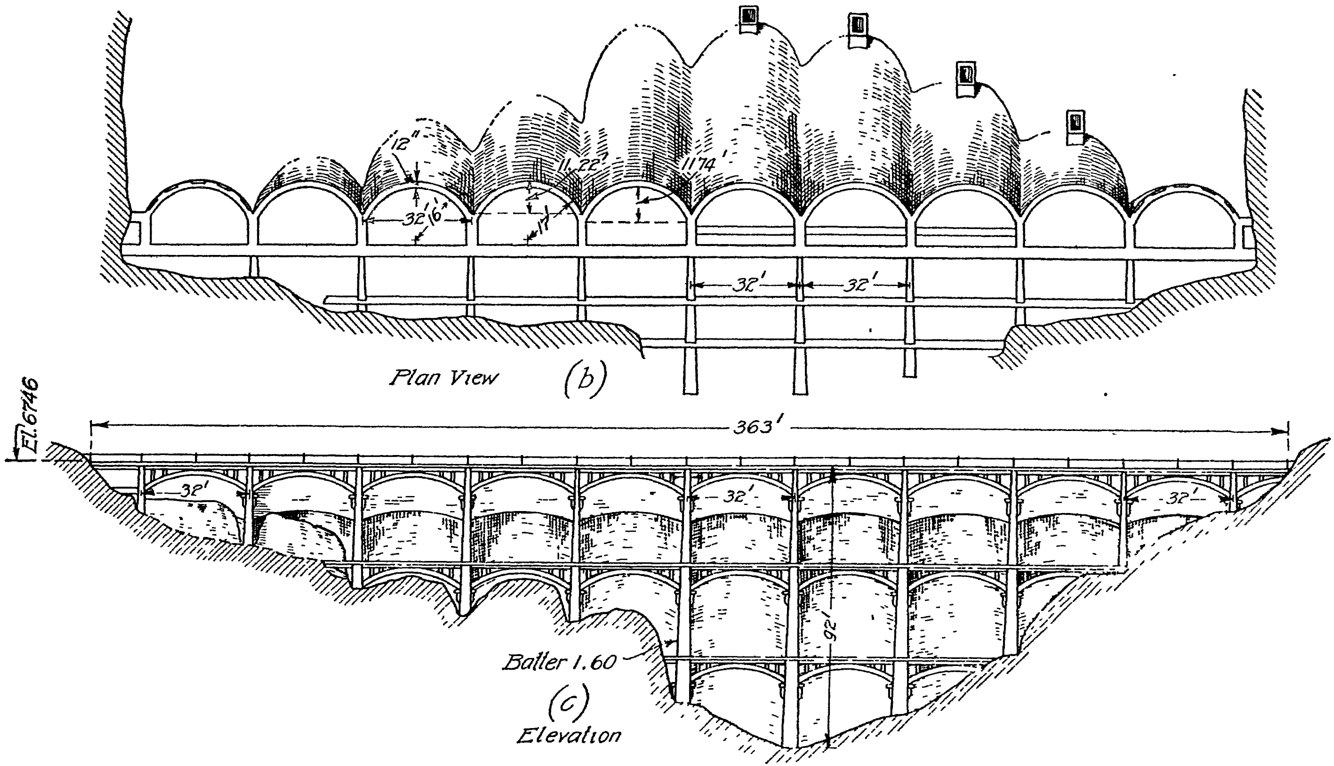


Fig 76. Plan and Sectional Elevation of Big Bear Valley Dam

At heel 3.9 tons, at center 2.7, and at toe, 4.2 tons.

The stress in the arch under a head of 38 feet comes to 11.5 tons. Thus the stresses in stage 2 are higher than is the case with stage 1.

At the end of a series of these scallop arches near either abutment the thrust of the arch resolved axially with the weir has to be met either by tying the last two arches by a cross wall and reinforcing rods, or abutting the arch on an abutment supported by wall or a length of solid dam. This design would, it is considered, be improved if the versed sine of the arcs were made somewhat greater, as the arches are too flat near the crest.

The following remarks bear on the curvature of the arch mentioned in section 101. When a segmental arch is inclined, the spring line is at a lower level than the crown, consequently the water pressure is also greater at that level. But the thickness should vary with the pressure which it does not in this case. This proves the advisability of making the circular curvature horizontal, then a section at right angles to the inclined spring line will be an ellipse, while a horizontal section will be a segment of a circle. The reverse occurs with arches built in the ordinary way. There appears to be no practical difficulty in constructing forms for an inclined arch on this principle.

103. Big Bear Valley Dam. Fig. 76 is a plan and sectional elevation of the new Bear Valley reinforced concrete multiple arch dam which takes the place of the old single arch dam mentioned in section 83. The following description is taken from "Engineering News", from which Fig 78 is also obtained.

The new dam consists of ten arches of $30\frac{1}{2}$ feet, clear span at top, abutting on eleven buttresses. The total length of the dam is 363 feet on the crest; its maximum height from crest to base is 92 feet (in a pocket at the middle buttress only), although, as the elevation in Fig. 76 shows, the average height of the buttresses is much less than that figure. The water face of the structure and the rear edge of the buttresses are given such slopes as to bring the resultant of the water-pressure load and that of the structure through the center of the base of the buttresses at the highest portions of the dam, Fig. 79. The slope for the water face up to within 14 feet of the top is 36° 52′ from the vertical, and

from that point to the crest is vertical. The slope of the downstream edges of the buttresses is 2 on 1 from the bottom to the top, the vertical top of the face arches giving the piers a top width of 10 feet from the spring line to the back edge. The buttresses are 1.5 feet thick at the top and increase in thickness with a batter of 0.016 feet per foot of height or 1 in 60 on each side to the base for all heights. The arch rings are 12 inches thick at the top and down to the bend, from which point they are increased in thickness at the rate of 0.014 feet per foot to the base, or 1 in 72.5.

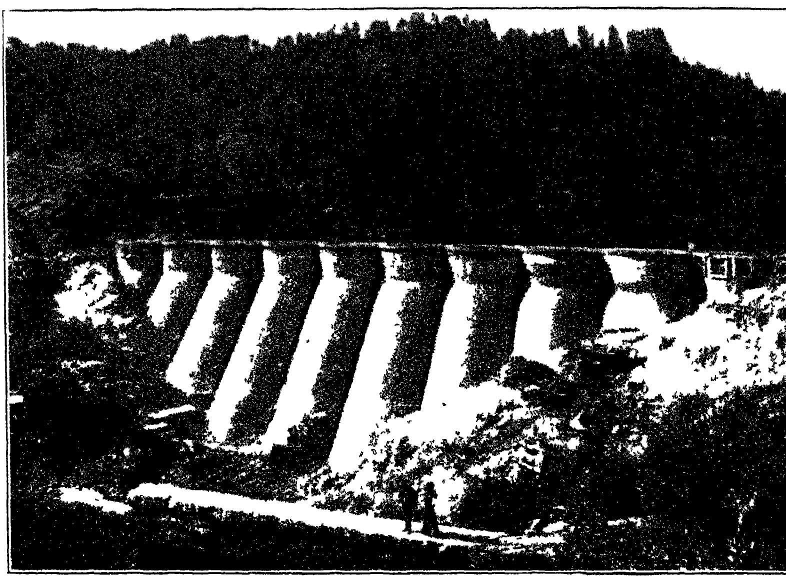


Fig 77. View of Big Bear Valley Dam with Old Dam Shown in Foreground

The arc of the extrados of the arch ring is 140° 08′ from the top to bottom the radius being maintained at 17 feet and the rise at 11.22 feet. The extrados is, therefore, a cylindrical surface uniform throughout, all changes in dimensions being made on the intrados of the arch. Thus at the top, the radius of the intrados is 16 feet, the arc 145° 08′, and the rise 11.74 feet. At 80 feet from the top, Fig. 79, the thickness of the arch ring will be 2.15 feet, the radius of the intrados 14.85 feet (the radius of extrados less the thickness of the wall), the arc 140° 48′ and the rise 10.59

feet. In all cases of arch-dam design the clear span, radius, and rise of the intrados decrease from the top downward.

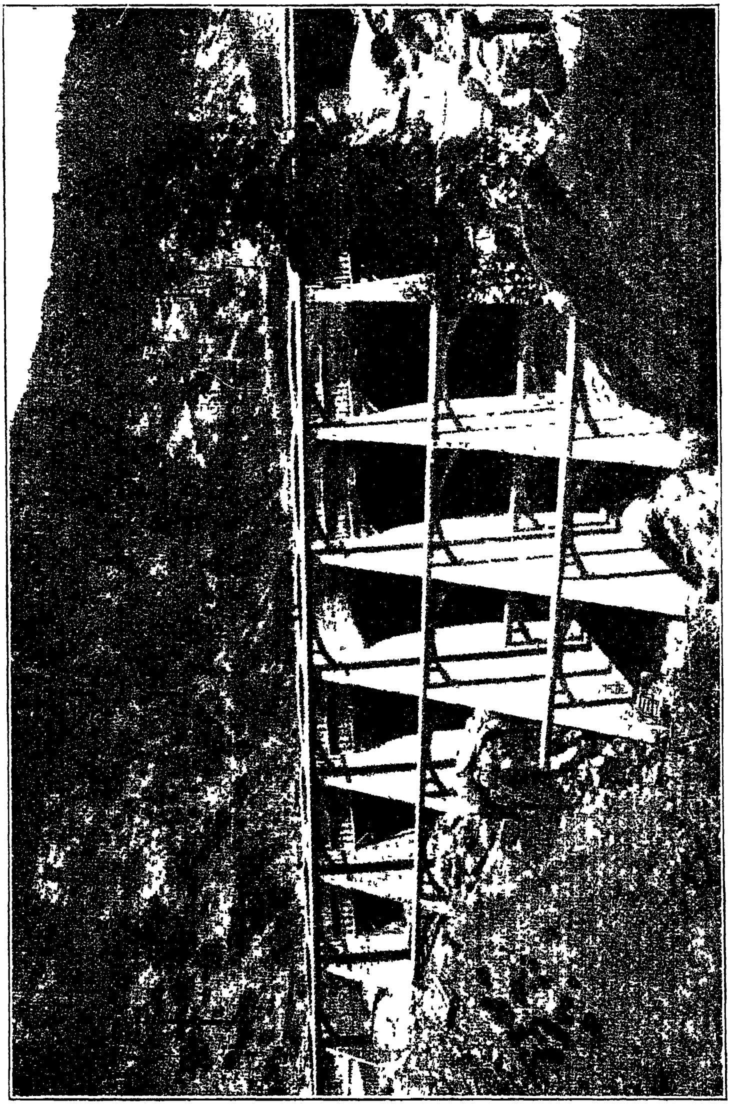


Fig 78 Back View of Big Bear Valley Dam, Showing Construction of Multiple Arches and Buttresses
Courtesy of "Engineering News"

Strut-tie members are provided between the buttresses to stiffen and take up any lateral thrusts that might be set up by seismic disturbances or vibrations, these consisting of T-beams

and supporting arches all tied together by heavy steel reinforcement. The T-beams are 12 inches thick and 2.5 feet wide, with a 12-inch stem, set on an arch 12 inches square at the crown and thickening to 15 inches toward the springing lines, with two spandrel posts on each side connecting the beam and arch, all united into one piece. There are provided copings for the arches and the tops of the buttresses with 9-inch projections, making the arch cope 2.5 feet wide and that on top of the buttresses 3 feet wide. The beam slab of the top strut members is built 4 feet wide to serve as an extra stiffener, as well as a comfortable footwalk across the dam. This footwalk is provided with a cable railing on both sides

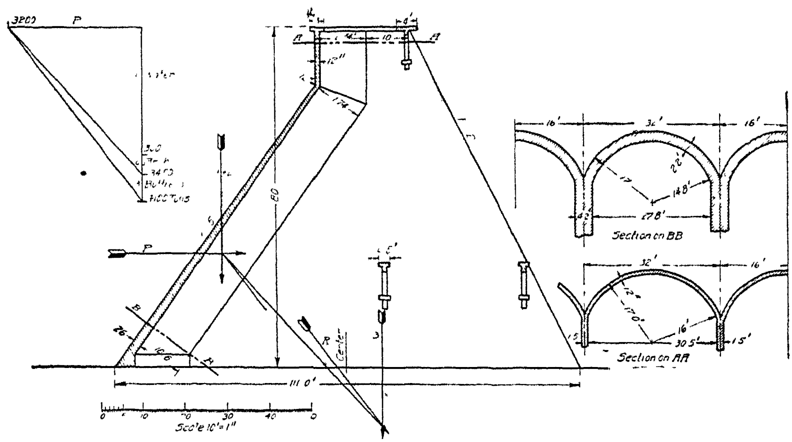


Fig 79 Profile and Sections of Big Bear Valley Dam

to make it a safe place upon which to walk. To add to the architectural effect of the structure, the arches of the strut members terminate in imposts, built as part of the buttresses. The struts are reinforced with twisted steel rods, all being tied together and all being continuous through the buttress walls. The ends entering the buttresses are attached to other reinforcement passing crosswise into the buttress walls, forming roots by which the stresses in the beams may be transmitted to and distributed in the buttress walls. The ends of the strut members are all tied onto the granite rock at both ends of the structure by hooking the reinforcement rods into drill holes in the rock. The buttresses are not reinforced, except to be tied to the arch rings and the strut members, their

shape and the loads they are to carry making reinforcement superfluous. The arch ribs are reinforced with $\frac{3}{4}$-inch twisted rods horizontally disposed 2 inches from the inner surface and variably spaced. These rods were tied to the rods protruding from the buttresses. For reinforcing the extrados of the arch ring ribs of $1\frac{1}{2}\times1\frac{1}{2}\times\frac{3}{16}$-inch angles were used, to which "ferro-inclave" sheets were clipped and used both as a concrete form for the outer face and a base for the plaster surface.

104. Stress Analysis. On Fig. 79 a rough stress analysis is shown for 80 feet depth of water. As will be seen the resultant R cuts the base just short of the center point. The value of N is estimated at 4100 tons, the area of the base $A=110\times4.2=460$ sq. feet whence $\frac{N}{A}=\frac{4100}{460}=9$ tons nearly, evenly distributed (m being taken as unity). The stress on the arch, 80 feet deep, neglecting its weight is $\frac{RHw}{b}-\frac{16\times80\times1}{2.2\times32}=20$ tons, nearly. This shows the necessity for the reinforcement provided to take $\frac{1}{2}$ or $\frac{3}{8}$ of this stress. The tangent of $\theta=\frac{P}{N}=\frac{3200}{4100}=.78.\quad\therefore\theta=39°.$

This is a large value, 35 degrees being the usual limit, 33 degrees better. If the arch thickness were doubled, reinforcement would not be necessary except near the crest and the additional load of about 320 tons would bring θ down to 35 degrees. If not, a greater inclination given to the arch would increase the load of water on the extrados. It is quite possible that a thicker arch without reinforcement would be actually cheaper. The downward thrust acting on the arch due to its own weight is on a different plane from the arch thrust. Its effect is to increase the unit stress to a certain extent, as is also the case with the combination of shearing and compressive stresses in the interior of a dam as explained in Part I. This increase can, however, be neglected. A considerable but undefined proportion of the water pressure near the base is conveyed to it and not to the buttresses; this will more than compensate for any increase due to vertical compression and consequently it can be ignored. The ribs connecting the buttresses form an excellent provision for stiffening them against buckling and vibration and are universally

employed in hollow concrete dams. The buttresses in this instance are not reinforced.

HOLLOW SLAB BUTTRESS DAMS

105. Description of Type. There is a class of dam and weir similar in its main principles to the arch buttress type which is believed to have been first introduced by the Ambursen Hydraulic Construction Company of Boston. In place of the arch an inclined flat deck is substituted, which has necessarily to be made of reinforced concrete. For this reason, the deck slabs cannot exceed a moderate width, so numerous narrow piers take the place of the thick buttresses in the former type. A further development is a thin deck which covers the downstream ends of the buttresses or piers, forming a rollway. The enclosed box thus formed is occasionally utilized as a power house for the installation of turbines, for which purpose it is well suited.

The inclination given to the flat deck is such that the incidence of the resultant (R F.) will fall as near the center of the base as possible and at the same time regulate the inclination of the resultant to an angle not greater than that of the angle of friction of the material, i.e., 30 degrees with the vertical. By this means any tendency to slide on the foundation is obviated.

Ellsworth Dam an Example. A good example of this style of construction is given in Fig. 80 of the Ellsworth dam in Maine. In this design the inclination of the deck is 45° or very nearly so; the piers are 15 feet centers with widened ends, so that the clear span of the concrete slabs is 9′ 1″ at the bottom.

The calculations necessary to analyze the thickness of the slabs and the steel reinforcement at one point, viz, at *El.* 2.5, will now be given. In this case the pressure of water on a strip of the slab, one foot wide, the unsupported span of which is 9′ 1″, is Hlw. Here $H=67$ feet and w is $\frac{1}{32}$ ton per cubic foot; therefore, $W=67\times 9.1\times\frac{1}{32}=19$ tons. To this must be added the weight of the slab. As this latter lies at an angle with the horizontal its weight is partly carried by the base and is not entirely supported by the piers. The diagram in Fig. 80c is the triangle of forces. The weight of slab w is resolved in two directions, a and b, respectively, parallel

and normal to face of slab. The angle being 45 degrees, $a=b=\frac{w}{\sqrt{2}}$. Consequently the thickness, 37 inches, can be considered as reduced to $\frac{3.1}{1.4}=2.2$ feet. The portion of the weight of the slab carried to the piers will, therefore, be $9.1\times2.2\times\frac{3}{40}=1.5$ tons, the weight of the concrete being assumed at the usual value of 150 pounds per cubic foot. The total distributed load in the strip will then be $19+1.5=20.5$ tons.

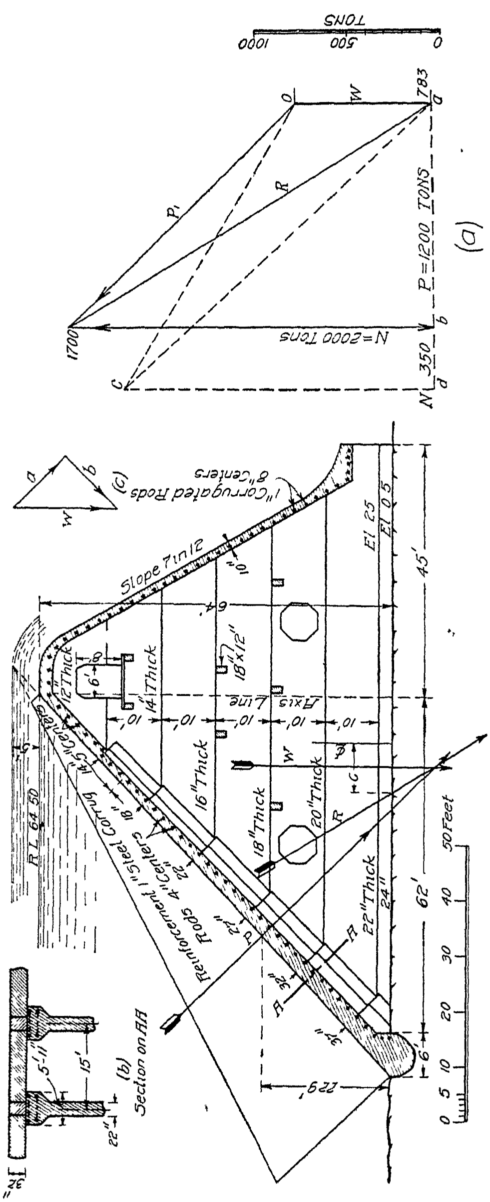


Fig 80 Profile and Force Diagram of Ellsworth Dam

Now the moment of stress on a uniformly loaded beam with free ends is $\frac{Wl}{8}$, or $M=\frac{20.5\times109}{8}=279$ inch-tons.

This moment must be equaled by that of the resistance of the concrete slab.

106. Formulas for Reinforced Concrete. For the purpose of showing the calculations in detail, some leading formulas connected with reinforced beams and slabs will now be exhibited:

$$bd^2=\frac{M_s}{f_s pj} \quad \textbf{(26)}$$

or, approximately,
$$bd^2=\frac{M_s}{\frac{7}{8}f_s p} \quad \textbf{(26a)}$$

$$bd^2=\frac{M_c}{\frac{1}{2}f_c kj} \quad \textbf{(27)}$$

or, approximately,
$$bd^2=\frac{M_c}{\frac{1}{6}f_c} \quad \textbf{(27a)}$$

From these are found d, the required thickness of a slab up to centroid of steel, or M_s+M_c the bending moments, in which b is width of beam in inches; d depth of centroid of steel below top of beam; M_c and M_s symbolize the moments of resistance of the concrete and steel, respectively; f_s safe unit fiber stress in steel, 12,000 to 16,000 lb., or 6 to 8 tons per square inch; f_c safe extreme stress in concrete 500, 600, or 650 lb., or .25, .3, or .325 ton per square inch; p steel ratio, or $\frac{A}{bd}$

$$\text{Ideal steel ratio } p=\frac{n}{2(r^2+rn)} \quad \textbf{(28)}$$

A area of cross-section of steel; k ratio of depth of neutral axis below top to depth of beam

$$k=\sqrt{2pn+(pn)^2}-pn \quad \textbf{(29)}$$

j ratio of arm of resisting couple to d

$$j=(1-\tfrac{1}{3}k) \quad \textbf{(30)}$$

n ratio $\frac{E_s}{E_c}$, E_s and E_c being the moduli of elasticity, ordinary values 12 to 15; r ratio $\frac{f_s}{f_c}$. As $p=\frac{A}{bd}$, when reinforced slabs are analyzed, formulas (26) and (27) can be transposed as below.

From (26) $M_s=f_sAjd$ **(31)**

From (26a) $M_s=\frac{7}{8}f_sAd$ Approximate **(31a)**

From (27) $M_c=\frac{1}{2}f_ckjbd^2$ **(32)**

From (27a) $M_c=\frac{f_c\,bd^2}{6}$ Approximate **(32a)**

In the case under review the reinforcement consists of three one inch square steel rods in each foot width of the slab. Using the

approximate formulas (**31a**) and (**32a**), $f_s=8$ tons, $f_c=.3$ ton, $d=35$ inches and $b=12$ inches; then

$$M_s=8\times3\times\frac{7}{8}\times35=735 \text{ inch-tons}$$

$$M_c=\frac{3}{10}\times\frac{1}{6}\times420\times35=735 \text{ inch-tons}$$

the results being identical. As already noted the moment of stress is but 279 inch-tons. The end shear may have governed the thickness. Testing for shear the load on a 12-inch strip of slab is 20.5 tons of which one-half is supported at each end. Allowing 50 lb., or .025 ton, as a safe stress, the area of concrete required is $10.25\div.025=410$ square inches the actual area being $37\times12=444$ square inches.

107. Steel in Fore Slope. The reinforcement of the fore slope is more a matter of judgment than of calculation, this deck having hardly any weight to support, as the falling water will shoot clear of it. The piers are not reinforced at all, nor is it necessary, as the stresses are all compressive and the inclination of the upstream deck is such that the resultant pressure makes an angle with the vertical not greater than that of friction, i.e., 30 degrees. Fig. 80a is a force diagram of the resultant forces acting on the base at *El.* 0.00. The total weight of a 15-foot bay is estimated at 783 tons while that of P, the trapezoid of water pressure, is 1700 tons. The force line P in Fig. 80 drawn through the c.g. of the water pressure area intersects the vertical force W below the base line. From this intersection R is drawn upward parallel to its reciprocal in the force polygon, cutting the base at a point some 9 feet distant from the center point.

The maximum stress will occur at the heel of the base. $A=107\times2=214$ sq. ft.; $\frac{N}{A}=\frac{2000}{214}=9.34$ tons; q being 9 ft., $m=\frac{107+54}{107}=1.5$ and $s=9.34\times1.5=14$ tons. Formula (9), Part I. The horizontal component of $P=1200$ tons. The base being 2 ft. wide, $s_s=\frac{1200}{2\times107}=5.6$ tons; therefore by formula (10), Part I, $c=7+\sqrt{49+31.4}=16.5$ tons, a decidedly high value. The usual limit to shearing stress is 100 lb. per sq. inch, equivalent to 7.2 tons per sq. ft., reinforcement is therefore not necessary and is not provided.

There appears to be no reason why a steeper slope should not have been given to the deck so as to bring the center of pressure up

to the center of the base and thus reduce the unit stress. Possibly a higher river stage has been allowed for. The position of W as well as the weight of the structure were obtained from the section given in Schuyler's Reservoirs. Fig. 80 is of the so-termed "Curtain" type of dam. The "Half Apron" type, Fig. 82c, is sometimes used for overfalls, the main section of Fig 82 illustrating the "Bulkhead" type.

108. Slab Deck Compared with Arch Deck Dam. The Ambursen dam, wherever the interior space is not required for installation of turbines, is undoubtedly a more expensive construction than the multiple arch type. This fact has at last been recognized and in one of the latest dams erected, scallop arches were substituted for the flat deck, thus obviating the expense of reinforcement. By

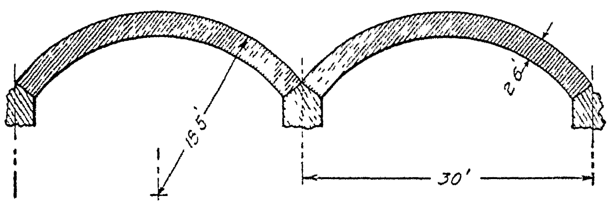


Fig 81 Section of Arch with 30-Foot Span

increasing the width of the spans, the piers, being thicker in like proportion, will be in much better position for resisting compressive stress, as a thick column can stand a greater unit stress than a thin one. Another point in favor of the arch is that the effective length of the base of the piers extends practically to the crown of the arch. The arch itself need not be as thick as the slab. Owing to the liquid radial pressure to which it is subjected it is in a permanent state of compression and does not require any reinforcement except possibly at the top of the dam. Here the arch is generally widened, as in the case of the Ogden dam, Fig. 73, and thus greatly stiffened at the point where temperature variations might develop unforeseen stresses.

Fig. 81 is a sketch illustrative of the saving in material afforded by doubling the spans from 15 to 30 feet and conversion to multiple arch type. The radius of the extrados of the arches is 18.5 ft. H is 67 at elevation 2.50 and $w=\frac{1}{32}$ ton; hence the thickness of the arch by formula (21) (s_1 being taken as 15 tons), will be

$$b=\frac{RHw}{s}=\frac{18.5\times67\times1}{32\times15}=2.6 \text{ feet}$$

It is thus actually thinner than the reinforced slab of one-half the span, or 15 feet. The greater length of the arch ring over that of the straight slab is thus more than compensated. The area of the arch, counting from the center of the pier, is $35 \times 2.6 = 91$ square feet, that of the slab is $30 \times 3.1 = 93$, that of the bracketing at junction with the piers, 13, giving a total of 106 square feet. The saving

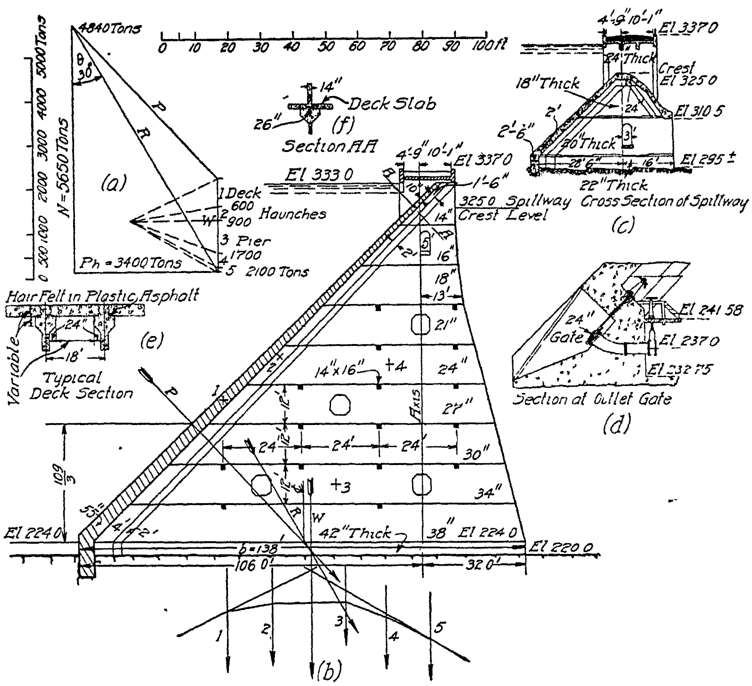


Fig 82. Profile and Detailed Sections of Guayabal Dam, Porto Rico

due to decreased length of the piers is 25 square feet. Thus in the lower part of the dam over 40 cubic feet per 30′ bay per foot in height of concrete is saved, also all the steel reinforcement. If a rollway is considered necessary in the weir, the deck could be formed by a thin reinforced concrete screen supported on I-beams stretching across between the piers.

109. Guayabal Dam. Fig. 82 is a section of the Guayabal dam recently constructed in Porto Rico, its height is 127 feet and it is on a rock foundation. The following are the conditions govern-

ing the design; maximum pressure on foundation 10 tons per square foot; compression in buttresses 300 pounds per square inch or 21.6 tons per square foot; shear in buttresses 100 pounds per square inch, or 7.2 tons per square foot; shear in deck slabs 60 pounds, or .03 ton per square inch; f_c for deck slabs 600 pounds or .3 ton per square inch; f_s for deck slabs 14,000 pounds, or 7 tons per square inch.

The concrete in the slabs is in the proportion of 1:2:4, in the buttresses 1:3:6; $n=\frac{E_s}{E_c}$ is taken as 15 and $r=\frac{f_s}{f_c}=23.3$. The deck slab is 55 inches thick at *El.* 224, d is taken as 53, allowing 2 inches for covering the steel, bd or the area of the section one foot wide $=53\times 12=636$ square inches. Now A the area of the steel $=pbd$. By formula (**28**), $p=\frac{n}{2(r^2+rn)}=\frac{15}{2(23.3)^2+23.3\times 15}=.01044$, hence the required area of steel will be $636\times .01044=6.64$ square inches, provided d is of the correct value. The calculation will now be made for the thickness of the slab which is actually 55 inches. The load on a strip 12 inches wide is

$$\text{Water pressure } \frac{109\times 13}{32}=44.3 \text{ tons}$$

To this must be added a portion of the weight of the slab which latter amounts to $\left(\frac{13\times 55}{12}\right)\times\left(\frac{3}{40}\right)=4.5$ tons. Of this $\frac{4.5}{\sqrt{2}}=3.2$ tons must be added to the 44.3 tons above, $44.3+3.2=47.5$ tons. The bending moment M is $\frac{WL}{8}=\frac{47.5\times 13\times 12}{8}=927$ inch-tons. The depth of the slab can be estimated by using formulas (**26**) or (**27**) or the approximate ones (**26a**) and (**27a**). For the purpose of illustration, all four will be worked out. First the values of k and j will be found by formulas (**29**) and (**30**).

$$k=\sqrt{.313+.0245}-.156=.582-.156=0.426$$

$$j=\left(1=\frac{k}{3}\right)=1-.142=.858$$

$$\text{By formula (26), } d^2=\frac{M_s}{12f_s pj}=\frac{927}{12\times 7\times .0104\times .858}=1234$$

$$\therefore \qquad d=\sqrt{1234}=35.07 \text{ inches}$$

By formula (**27**), $d^2=\dfrac{2M_c}{12f_ckj}=\dfrac{927\times 2}{12\times .3\times .426\times .858}=1406$

$$\therefore \qquad d=\ 1406=37.5 \text{ inches}$$

Now the approximate formulas will be used. By (**26a**)

$$d^2=\frac{8\times 927}{7\times 12\times 7\times .0104}=\frac{1854}{1.53}=1210$$

$$\therefore \qquad d=\sqrt{1210}=34.8 \text{ inches}$$

by (**27a**)

$$d^2=\frac{6\times 927}{12\times .3}=\frac{5562}{3.6}=1542$$

$$\therefore \qquad d=\sqrt{1542}=39.3 \text{ inches}$$

The approximate formulas (**26a**) and (**27a**) give higher results than (**26**) and (**27**). The result to select is 37.5 inches, formula (**27**), which is higher than by (**26**). The depth of beam would then be 40 or 41 inches. It is actually 55. This discrepancy may be due to the water having been given a s.g. in excess of unity, owing to the presence of mud in suspension, say of 1.3 or 1.5, or shear is the criterion.

The corresponding steel area will be $A=pbd=.0104\times 12\times 37.5$ $=4.7$ square inches. $1\frac{7}{16}''$ round rods spaced 3 inches would answer. With regard to direct shear on the slab, W as before $=47.5$ tons of which half acts at each pier, viz, 23.7 tons. The safe resistance is $bd\times S_s=12\times 55\times .03=20$ tons, nearly. The shear $=\dfrac{23.7}{12\times 55}=.036$ ton $=72$ pounds per square inch. This figure exceeds the limit of 60 pounds. The deficiency is made up by adding the shear of the steel rods. The sectional area of this reinforcement is 4.7 square inches the safe shearing of which is over 20 tons. These rods are usually turned up at their ends in order to care for the shear.

Shear in Buttresses. With regard to shear in the buttresses, the horizontal component of the water pressure as marked on the force diagram is 3400 tons. The area of the base of the buttress at *El.* 224 is $138\times 3.2=441.6$, the shearing stress or s_s then $=\dfrac{3400}{441.6}=8$ tons per square foot, nearly. The allowable stress being only 7.2 tons the difference will have to be made good by reinforcing rods of which two of $\frac{3}{4}$-inch diameter would suffice.

Now with regard to compressive stresses on the buttresses the graphical working shows that the resultant R strikes the base at *El.* 224 almost exactly at the center, the angle θ also is 30 degrees. The value of N is 5650 tons; s_1 the mean and s the maximum stress will both equal $\frac{N}{A}$; and A, the area of the base, equals $138 \times 3.2 = 442$ sq. ft.; therefore, $s = \frac{5650}{442} = 12.78$ tons. The compression on the foundation itself, which is 4 feet lower will not be any less for, although the base width is greater, N as well as P are also increased. Thus the pressure on the foundation is in excess of the limit and widening to a further extent is required.

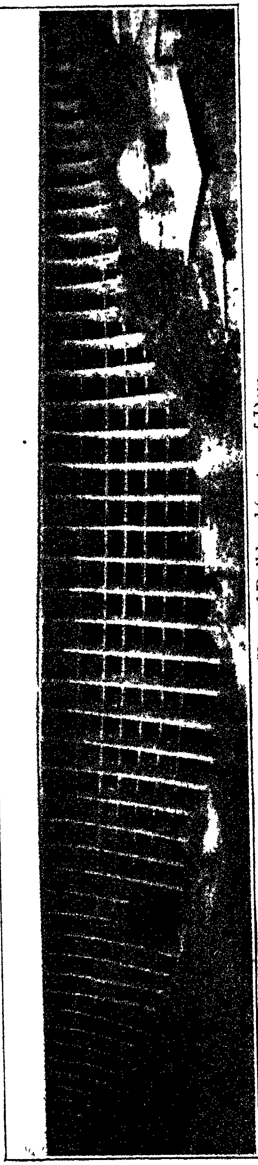


Fig. 83. Rear View of Bulkhead Section of Dam

This section has 51 spans with 18-foot centers, making a total length of 918 feet

The maximum internal stress c, in the buttress at *El.* 224, will be by formula (10), Part I, $\frac{1}{2}s + \sqrt{\frac{s^2}{4} + s_s^2}$. Here $s = 12.8$ and s_s as we have seen is 8 tons, therefore, $c = 6.4 + \sqrt{\frac{164}{4} + 64} = 16.6$ tons. The limit compression in the buttress is 300 pounds per square inch, or 21.6 tons per square foot.

In the bulkhead portion of the dam, shown in Fig. 82b, every pier is run up 14 inches thick through the deck to form a support for a highway bridge, the spans of which are therefore 16 feet 10 inches in the clear; the roadway is carried on slabs which are supported by arches of reinforced concrete. The buttresses are laterally supported by several double reinforced sway beams, 16″×14″, and below the crest a through roadway is provided. The spillway section is shown on Fig. 82c. The

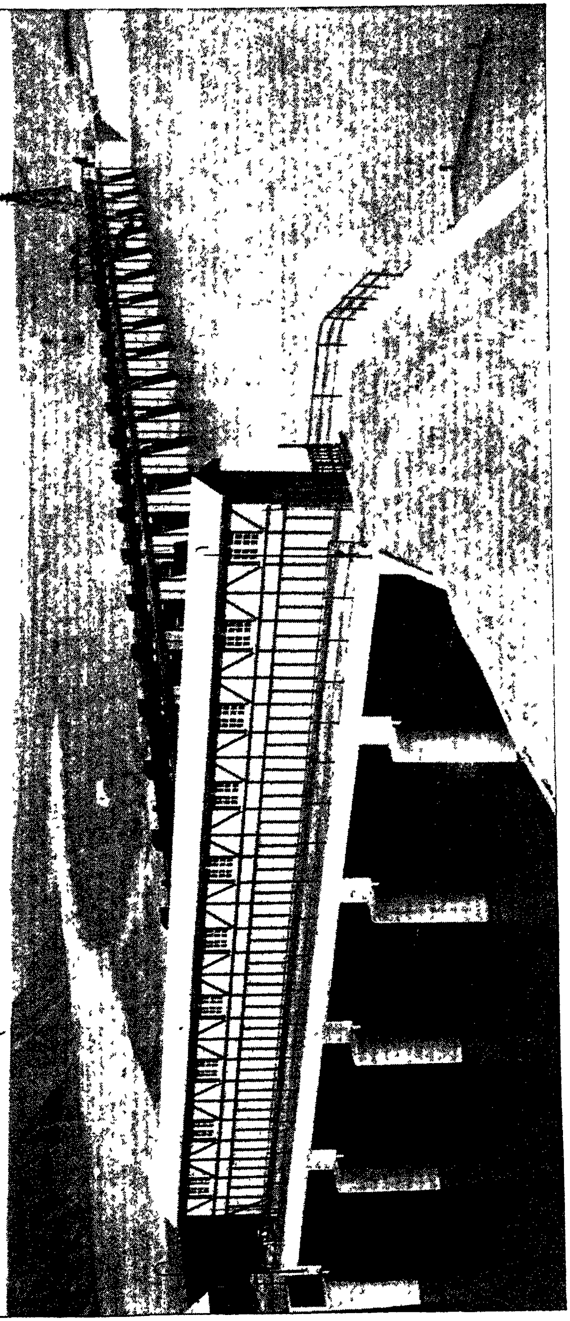


Fig. 84. View of Bassano Dam over the Bow River Taken Just after Water Was Turned into Can

ground level is here on a high bench at *El.* 295. The crest being *El.* 325, the fall is 30 feet. The spillway is of the "half apron type". The roadway here is carried on four reinforced concrete girders, a very neat construction; the piers are run up every alternate span and are therefore at 36-foot centers; they are beveled on both faces to reduce end contraction. The spillway will pass 70,000 second-feet; its length is 775 feet.

The bulkhead section of the dam (see also Fig. 83) has 51 spans of 18-foot centers, total length 918 feet; that of the spillway consists of 21 spans of 36-foot centers. The whole length is 1674 feet. The depth of the tail water is not known, it would probably be about 20 feet and its effect would be but trifling. This is one of the largest hollow dams ever constructed. The arrangement of the haunches or corbels of the buttresses is a better one than that in the older work of Fig. 80.

110. Bassano Dam. Another important work is the Bassano dam illustrated in Figs. 84 and 85. This is an overfall dam built over the Bow River at the head of the eastern section of the Canadian Pacific Railway Company's irrigation canal and is estimated to pass 100,000 second-feet of water at a depth of 14 feet. Though not so high nor so long as the Guayabal dam it presents several features of interest. First its foundations are on a thick blanket of clay some twelve feet deep which overlies boulders and gravel. This material is very hard blue clay of excellent quality. The great advantage of this formation, which extends over 1000 feet upstream from the work, is that it precludes all uplift, or very nearly so, consequently no special precautions have to be adopted, such as a long apron to ensure length of percolation, as would be necessary in case of a foundation composed of porous and loose materials. It has also disadvantages. The allowable pressure on the clay is limited to $2\frac{1}{2}$ tons per square foot. This influences the design necessitating a wide spread to the buttresses, laterally as well as longitudinally. The whole of the dam is an overfall and the general arrangements are very similar to those prevailing at Guayabal. The hearth or horizontal fore apron, a provision not necessary in the last example, is at *El.* 2512. The crest is at 2549.6 a height of 37.6 feet above the apron and corresponds with the level of the canal intake floor. Water is held up to eleven feet above crest level by draw gates

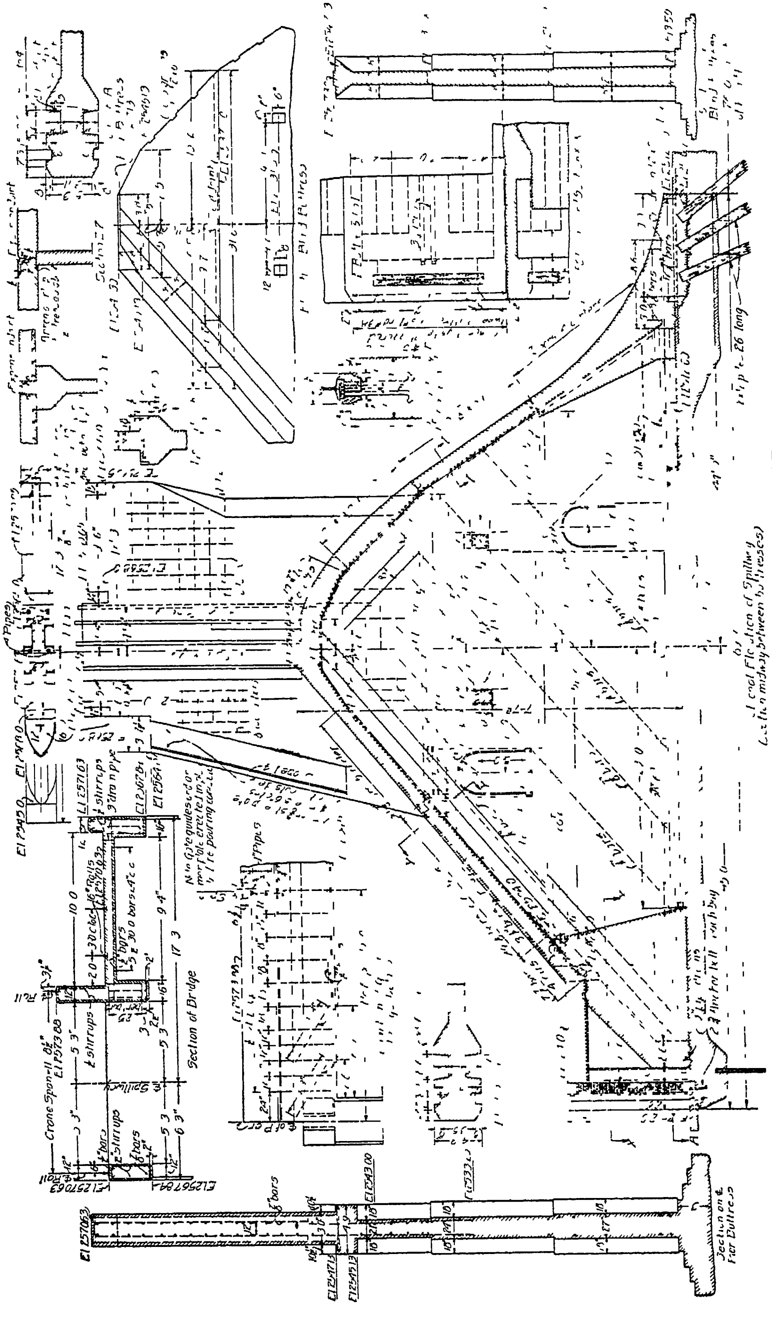


Fig 85. Profile and Sections of Bassano Overfall Dam

eleven feet high, and this full supply level is three feet below that of the estimated afflux, which is fourteen feet above the crest.

For overturning moment the water-pressure area will be a truncated triangle with its apex at afflux level plus the height h or $1.5\,\frac{v^2}{2g}$ to allow for velocity of approach, as explained in section 57, Part I. This, in the Bow River with a steep boulder bed will be about 12 feet per second; h therefore will equal $1.5\times\frac{144}{64}=3.4$ feet and the apex of the truncated triangle will be at a point $14+3.4=17.4$ feet above the crest level. The depth of the tail water at full flood is not known, the ratio $\frac{d}{D}$ with a steep bed slope will not be under .5, consequently with $d=14$, D will have a value of about 25 to 28 feet, d being depth over crest and D that of tail water. The overturning moments direct and reverse can be represented by the cubes of the depths up- and downstream and the unbalanced moment by their difference. The upstream head is $37.0+14+3.4=55$ feet and the downstream head say 25 feet. Their cubes are 166,375 and 15,625 the difference being 150,750, thus the reverse pressure will not have much effect in assisting the stability of the structure. The corresponding representative moment when water is held up to 11 ft. above crest will be $49^3=117{,}649$, supposing the tail channel empty. This quantity is less than the 150,750 previously stated, consequently the afflux level is that which has to be considered when estimating the overturning moment. In the case of direct water pressure on the deck slabs, the acting head at full flood will be the difference of the flood level up- and downstream, which is 30 feet, as the tail water is allowed access to the rear of the deck slabs. This is less than the head, 49 feet, which exists when the gates are closed and water is held up to canal full supply, i.e., to *El.* 2560.6, consequently the head that has to be considered is that at this latter stage.

Analysis of Pressures on Bassano Dam. With this data the design can be analyzed, the procedure being identical with that explained in the last example, excepting that the reverse pressure might be taken into account as it will modify the direction and incidence of R in a favorable sense though not to any great extent. The limit stresses are those given in the last example with the fol-

lowing additions: Footings, compression in bending, 600 pounds per square inch, shear, 75 pounds per square inch.

Some explanation will now be given of the method of calculation of the footing to the buttresses and the Guayabal dam will be referred to, as the pressures on the base of the buttresses are known quantities. In section 109 the value of N is 5650 tons and $\frac{N}{b}=\frac{5650}{138}=41$ tons, nearly. This is the unit pressure per foot run on the base of the buttress. Supposing the limit pressure on the foundation was fixed at 3 tons per square foot, then the requisite base width of the footing would be $\frac{41}{3}=13.7$ feet. The footing consists of two cantilevers attached to the stem of the buttress. The bending moment M at the junction with the buttress of a strip 1 foot wide will be $\frac{Wl}{2}$. The buttress being 3.2 feet wide the projecting length of footing on each side will be $\frac{13.7-3.2}{2}=5.25$ feet.

The reaction on a strip one foot wide will be $5.25\times3\times1=15.75$ tons. The moment in inch-tons about the edge of the section of the buttress will be $\frac{12Wl}{2}=\frac{15.75\times5.25\times12}{2}=498$ inch-tons. According to formula (27a), $bd^2=\frac{6M}{f_c}$. $\therefore d=\sqrt{\frac{6\times498}{12\times.3}}=\sqrt{830}=$ 28.8 inches. Then $bd=28.8\times12=346$ and A the area of the steel at the base will be $pbd=.0104\times346=3.61$ inches, this in a 12-inch wide strip will take $1\frac{1}{4}$-inch bars 4 inches apart. When the weight on the buttress is considerable the depth of footing slab thus estimated becomes too great for convenience. In such cases, as in Fig. 85, the beam will require reinforcement in compression at the top. This complicates the calculation and cases of double reinforcement are best worked out by means of tables prepared for the purpose. The footings shown in Fig. 85, were thus double reinforced, in fact through bars were inserted at each step, the lower being in tension the upper ones in compression. The lower bars were continuous right through the base of the dam. This reinforcement of the footing is not shown on the blue print from which Fig. 85 is derived.

111. Pressure on Foundation Foredeck. A great many

Ambursen dams have been constructed on river beds composed of boulders and gravel, which require a pressure limit of about 4 tons per square foot. This can always be arranged for by widening the footing of the pier buttresses, the same can of course be done with arch buttressed dams. The base of the arch itself can be stepped out in a similar manner. In the Bassano dam the sloping fore deck is unusually thick and is heavily reinforced in addition; this is done with the idea of strengthening the structure against shock from ice, as well as from the falling water, and with the further idea of assisting the buttresses in carrying the heavy load of the piers and superstructure. It is doubtful if any calculations can well be made for this; it is a matter more of judgment than of estimation.

Buttresses. As with the Guayabal spillway, every alternate buttress is run up to form the piers of the superstructure, which latter consists of a through bridge which carries the lift gear for manipulating the draw gates. The so-termed blind buttresses—that is, those that do not carry a pier—are of thinner section and are apparently not reinforced. Both kinds of buttresses have cross-bracing as shown on the profile. In hollow dams the location of the center of pressure moves with the rise of water from the heel toward the center within the upstream half of the middle third. In solid dams, on the other hand, the movement is along the whole of the middle third division, consequently in hollow dams there is no tendency to turn about the toe as with solid dams, rather the reverse, namely, to upset backward. This latter tendency must cause tension in the buttresses which the cross-bracing is intended to care for.

Baffles. As noted already in section 66, baffles have been built on the curved bucket with the object of neutralizing this mischievous arrangement which it is hoped will soon become as obsolete in western practice as has long been the case in the East.

Hearth and Anchored Apron. The dam is provided with a solid horizontal fore apron or hearth 76 feet long and beyond this the device of an anchored floating apron of timber 30 feet in length has been added. The apron is undoubtedly too short and should have been made 100 feet or 2 $(H+d)$ in length, with cribbed riprap below it. The wooden sheet piling in the rear of the work is considered to be worse than useless; it merely breaks up the good clay

blanket by cutting it in two. A wide solid curtain of concrete, not so deep as to penetrate the clay blanket, would have been a superior arrangement. The inclined piling below the bucket is provided to guard against sliding. This dam is provided with a number of sluice openings. Their capacity is such that one half will pass ordinary floods, allowing the other half of the dam to be cut off from the river by sheet piling during construction. On completion of the work these sluices were all closed from the inside by slabs of concrete deposited in position.

SUBMERGED WEIRS FOUNDED ON SAND

112. Description of Type. There is a certain type of drowned or submerged diversion weir which is built across wide rivers or streams whose beds are composed of sand of such depth that a solid foundation on clay is an impossibility. Consequently, the weir has to be founded on nothing better than the surface of the river bed, with perhaps a few lines of hollow curtain walls as an adjunct. Of this class of weir but one is believed to have been constructed in the United States, viz, the Laguna weir over the Colorado River at the head of the Yuma irrigation canals.

This type originated in India and in that country are found numerous examples of weirs successfully constructed across very large rivers of immense flood discharge. For instance, the Godaveri River in Southern India has a flood discharge of 1,200,000 second-feet and the weirs across it are nearly $2\frac{1}{2}$ miles in length. Not only is the length great, but as will be seen, the width has to be very considerable. The Okhla weir, Figs. 101 and 102, situated on the Jumna below the historic city of Delhi is 250 feet wide and $\frac{3}{4}$ mile long. The height of these submerged weirs is seldom over 12 feet, their rôle being purely diversion, not storage. No doubt more of this type of low diversion weirs will in the future have to be constructed in the United States or in Mexico, so that a knowledge of the subject is a necessity for the irrigation engineer.

Principles of Design. The principles underlying the successful design of these works are a comparatively recent discovery. Designs were formerly made on no fixed principles, being but more or less modified copies of older works. Fortunately some of these works failed, and it is from the practical experience thus gained that a

knowledge of the hydraulic principles involved has at last been acquired.

A weir built on sand is exposed not only to the destructive influences of a large river in high flood which completely submerges it, but its foundation being sand, is liable to be undermined and worked out by the very small currents forced through the underlying sand by the pressure of the water held up in its rear. In spite of these apparent difficulties, it is quite practicable to design a work of such outline as will successfully resist all these disintegrating influences, and remain as solid and permanent a structure as one founded on bed rock.

113. Laws of Hydraulic Flow. The principle which underlies the action of water in a porous stratum of sand over which a heavy impervious weight is imposed is analogous to that which obtains in

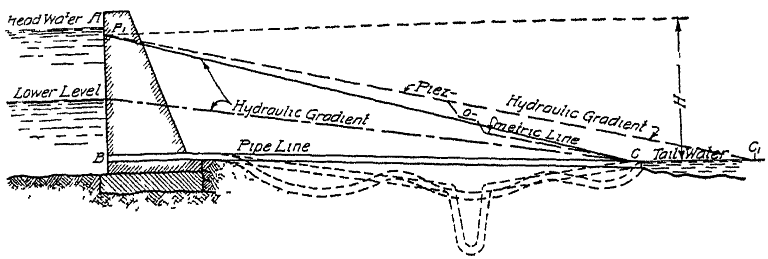


Fig 86 Diagram Showing Action of Water Pipe Leading Out of Reservoir

a pipe under pressure. Fig. 86 exemplifies the case with regard to a pipe line BC, leading out of a reservoir. The acting head (H) is the difference of levels between A_1 a point somewhat lower than A, the actual summit level and C the level of the tail water beyond the outlet of the pipe. The water having a free outlet at C, the line A_1C is the hydraulic gradient or grade line. The hydrostatic pressure in the pipe at any point is measured by vertical ordinates drawn from the center of the pipe to the grade line A_1C. The uniform velocity of the water in the pipe is dependent directly on the head and inversely on the frictional resistance of the sides of the pipe, that is, on its length. This supposes the pipe to be straight, or nearly so.

114. Percolation beneath Dam. We will now consider the case of an earthen embankment thrown across the sandy bed of a

stream, Fig. 87. The pressure of the impounded water will naturally cause leakage beneath the impervious earthen base. With a low depth of water impounded it may well be understood that such leakage might be harmless; that is, the velocity of the percolating under current would be insufficient to wash out the particles of sand composing the foundation of the dam. When, however, the head is increased beyond a safe limit, the so-termed piping action will take place and continue until the dam is completely undermined.

115. Governing Factor for Stability. The main governing factor in the stability of the sand foundation is evidently not the superimposed weight of the dam, as the sand is incompressible; although a load in excess of the hydraulic pressure must exercise a certain though possibly undefined salutary effect in delaying the disintegration of the substratum. However this may be, it is the

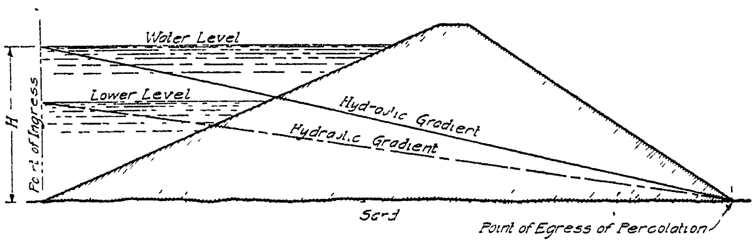


Fig. 87 Diagram Showing Effect of Percolation under Earthen Embankment across Stream

enforced length of percolation, or travel of the under current, that is now recognized to be the real determining influence.

In the case of a pipe, the induced velocity is inversely proportional to the length. In the case under consideration, the hydraulic condition being practically identical with that in a pipe, it is the enforced percolation through the sand, and the resulting friction against its particles as the water forces its way through, that effects the reduction of the velocity of the undercurrent, and this frictional resistance is directly proportional to the length of passage. In the case of Fig. 87, the length of enforced percolation is clearly that of the impervious base of the earthen dam. The moment this obstruction is passed the water is free to rise out of the sand and the hydrostatic pressure ceases.

116. Coefficient of Percolation. This length of enforced percolation or travel, which will be symbolized by L, must be some

multiple of the head H, and if reliable safe values for this factor can be found, suitable to particular classes of sand, we shall be enabled to design any work on a sand foundation, with perfect confidence in its stability. If the percolation factor be symbolized by c, then L, or the length of enforced percolation, will equal cH, H being the head of water. The factor c will vary in value with the quality of the sand.

Fig. 88 represents a case similar in every respect to the last except, instead of a dam of earth, the obstruction consists of a vertical wall termed the weir or drop wall, having a horizontal impervious floor attached thereto, which appendage is necessary to prevent erosion of the bed by the current of falling water.

At the stage of maximum pressure the head water will be level with the crest, and the level of the tail water that of the floor; consequently the hydraulic gradient will be HB, which is also the piezometric line and as in the previous case of the pipe line, Fig.

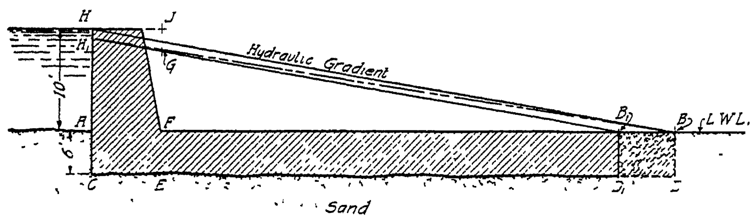


Fig. 88. Diagram Showing Design of Profile to Reduce Percolation

86, the ordinates of the triangle HAB will represent the upward hydrostatic pressure on the base of the weir wall and of the floor.

117. Criterion for Safety of Structure. The safety of the structure is evidently dependent on the following points:

First, the weir wall must be dimensioned to resist the overturning moment of the horizontal water pressure. This has been dealt with in a previous section. Second, the thickness, i. e., the weight of the apron or floor must be such that it will be safe from being blown up or fractured by the hydrostatic pressure; third, the base length, or that of the enforced percolation L, must not be less than cH, the product of the factor c with the head H. Fourth, the length of the masonry apron and its continuation in riprap or concrete book blocks must be sufficient to prevent erosion.

It is evident that the value of this factor c, must vary with the nature of the sand substratum in accordance with its qualities of

fineness or coarseness. Fine light sand will be closer in texture, passing less water under a given head than a coarser variety, but at the same time will be disintegrated and washed out under less pressure. Reliable values for c, on which the design mainly depends, can only be obtained experimentally, not from artificial experiments, but by deduction from actual examples of weirs; among which the most valuable are the records of failures due to insufficiency in length of percolation. From these statistics a safe value of the relation of L to H, the factor c, which is also the sine of the hydraulic gradient, can be derived.

118. Adopted Values of Percolation Coefficient. The following values of c have been adopted for the specified classes of sand.

Class I: River beds of light silt and sand, of which 60 per cent passes a 100-mesh sieve, as those of the Nile or Mississippi; percolation factor $c=18$.

Class II: Fine micaceous sand of which 80 per cent of the grains pass a 75-mesh sieve, as in Himalayan rivers and in such as the Colorado; $c=15$.

Class III: Coarse-grained sands, as in Central and South India; $c=12$.

Class IV: Boulders or shingle and gravel and sand mixed; c varies from 9 to 5.

In Fig. 88 if the sand extended only up to the level C, the length of percolation would be CD, the rise from D to B not being counted in. In that case the area of hydrostatic pressure acting beneath the floor would be the triangle HAB. As, however, a layer of sand from A to C interposes, the length will be ACD, and outline H_1B. The step HH_1 occurring in the outline is due to the neutralization of head symbolized by h, effected in the depth AC. Supposing AC to be 6 feet and the percolation factor to be 12, then the step in the pressure area, equal to h, will be $6 \div 12 = 6$ inches. The resulting gradient H_1B will, however, be flatter than 1 in 12; consequently the termination of the apron can be shifted back to B_1D_1, H_1B_1, being parallel to HB; in which case the area of hydrostatic pressure will be H_1AB_1. The pressure at any point on the base is represented by the ordinates of the triangle or area of pressure. Thus the upward pressure at E, below the toe of the drop wall, where the horizontal apron commences, is represented by the line

FG. Supposing the head HA to be 10 feet, then the total required length of percolation will be $cH=12\times10=120$ feet. This is the length ACD_1. The neutralization of head, h, effected by the enforced percolation between H and G is represented by GJ, and supposing the base width of the drop wall CE to be 9 feet, AC being 6 feet, $h=\frac{6+9}{12}=1\frac{1}{4}$ feet. The upward pressure FG is $(H-h)=10-1\frac{1}{4}$ $=8\frac{3}{4}$ feet.

The stepped upper line bounding the pressure area as has been noted in Part I, is termed the piezometric line, as it represents the level to which water would rise if pipes were inserted in the floor. It is evident from the above that when no vertical depressions occur in the line of travel that the piezometric line will coincide with the hydraulic gradient or virtual slope; when, however, vertical depressions exist, reciprocal steps occur in the piezometric line, which then falls below the hydraulic grade line. The piezometric line is naturally always parallel to the latter. The commencement of the floor at E is always a critical point in the design as the pressure is greatest here, diminishing to zero at the end.

119. Simplifying the Computations. In the same way that the water pressure is represented by the head producing it, the common factor w, or the unit weight of water, may also be eliminated from the opposing weight of the floor. The weight of the masonry, therefore, is represented by its thickness in the same way as the pressure, and if t be the thickness of the floor, $t\rho$ will represent its weight. Now the floor lies wholly below low water level. Consequently, in addition to the external hydrostatic pressure represented by H, due to the head of water upheld, there is the buoyancy due to immersion. The actual pressure on the base CD_1 is really measured by HC, not HA. Thus if a vertical pipe were inserted in the floor the water would rise up to the piezometric line and be in depth the ordinate of the pressure area plus the thickness of the floor. But it is convenient to keep the hydrostatic external pressure distinct from the effect of immersion. This latter can be allowed for by reduction in the weight of these parts of the structure that lie below L. W. L. See sections 52 and 53, Part I.

Effect of Immersion. When a body is immersed in a liquid it loses weight to the extent of the weight of the liquid displaced.

Thus the unit weight of a solid is $w\rho$. When immersed, the unit weight will be $w(\rho-1)$. As w is a discarded factor, the unit weight being represented only by ρ, the specific gravity, the weight of the floor in question will be $t(\rho-1)$ if immersed. We have seen that the hydrostatic pressure acting at F is $8\frac{3}{4}$ feet. To meet this the weight, or effective thickness, of the floor must be equal to $8\frac{3}{4}$ feet of water $+\frac{1}{3}$ for safety, or, in symbols, $t=\frac{H-h}{\rho-1}\times\frac{4}{3}$

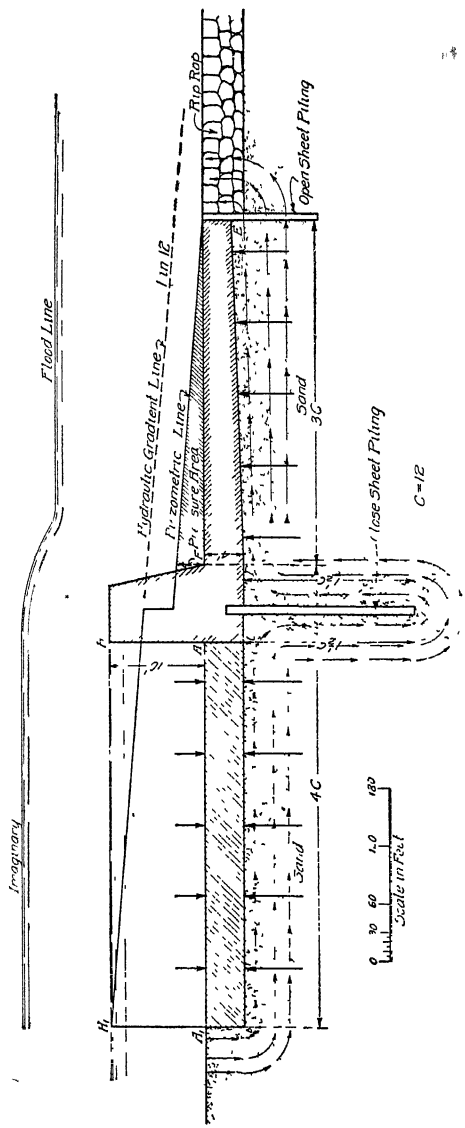


Fig 89. Profile of Weir on Sand Showing Method of Providing Requisite Length of Travel of Percolation

Assuming a value for ρ of 2, the thickness required to counterbalance the hydrostatic pressure will be

$$t=8\tfrac{3}{4}\times\frac{4}{3}=11.6 \text{ feet}$$

The formula for thickness will then stand:

$$t=\frac{4}{3}\left(\frac{H-h}{\rho-1}\right) \quad \textbf{(33)}$$

Uplift on Fore Apron. It is evident that in Fig. 88 the long floor is subjected to a very considerable uplift measured by the area HAB, the weight of the apron also is reduced in the ratio of $\rho:(\rho-1)$ as it lies below L.W.L., consequently it will have to be made as already noted of a depth of 11.6 feet which is a

quite impossible figure. The remedy is either to make the floor porous in which case the hydraulic gradient will fall below 1 in 12, and failure will take place by piping, or else to reduce the effective head by the insertion of a rear apron or a vertical curtain wall as has been already mentioned in section 53, Part I. In these submerged weirs on large rivers and in fact in most overfall dams a solid fore apron is advisable. The length of this should however be limited to absolute requirements. This length of floor is a matter more of individual judgment or following successful precedent than one of precise estimation.

The following empirical rule which takes into account the nature of the sand as well as the head of water is believed to be a good guide in determining the length of fore apron in a weir of this type, it is

$$L = 3\sqrt{cH} \qquad \textbf{(34)}$$

In the case of Fig. 89, the head is 10 feet and c is assumed at 12, consequently, $L = 3\sqrt{120} = 33$ feet, say 36, or $3c$. In Fig. 89 this length of floor has been inserted. Now a total length of percolation of 12 times the head, or 120 feet $= 10c$ is required by hypothesis, of this $3c$ is used up by the floor leaving $7c$ to be provided by a rear apron and curtain. Supposing the curtain is made a depth equal to $1\frac{1}{2}c$, this will dispose of 3 out of the 7 (for reasons to be given later), leaving 4 to be provided for by the rear apron, the length of which, counting from the toe of the weir wall, is made $4c$ or 48 feet. The hydraulic gradient starts from the point H' which is vertically above that of ingress A. At the location of the vertical diaphragm of sheet piling, a step takes place owing to the sudden reduction of head of 3 feet, the obstruction being $3c$ in length counting both sides. From here on, the line is termed the piezometric line and the pressure area is the space enclosed between it and the floor. The actual pressure area would include the floor itself, but this has been already allowed for in reduction of weight, its s.g. being taken as unity instead of 2.

The uplift on the weir wall is the area enclosed between its base and the piezometric line. In calculating overturning moment, if this portion were considered as having lost weight by immersion it would not quite fully represent the loss of effective weight due to uplift, because above the floor level the profile of the weir wall is

not rectangular, while that of the pressure area is more nearly so. The foundation could be treated this way, the superstructure above AF being given full s.g. and the uplift treated as a separate vertical force as was the case in Fig. 40, Part I.

120. Vertical Obstruction to Percolation. Now with regard to the vertical obstruction, when water percolates under pressure beneath an impervious platform the particles are impelled upward by the hydrostatic pressure against the base of the dam and also there is a slow horizontal current downstream. The line of least resistance is along the surface of any solid in preference to a shorter course through the middle of the sand, consequently when a vertical obstruction as a curtain wall of masonry or a diaphragm of sheet piling is encountered the current of water is forced downward and the obstruction being passed it ascends the other side up to the base line which it again follows. The outer particles follow the lead of the inner as is shown by the arrows in Fig. 89. The value of a vertical obstruction is accordingly twice that of a similar horizontal length of base. Valuable corroboration of the reliability of the theory of percolation adopted, particularly with regard to reduction of head caused by vertical obstruction, has been received, while this article was on the press, from a paper in the proceeedings of the American Society of Civil Engineers entitled "The Action of Water under Dams" by J. B. T. Coleman, which appeared in August, 1915. The practical value of the experiments, however, is somewhat vitiated by the smallness of the scale of operations and the disproportion in the ratio $H:L$ to actual conditions. The length of base of the dam experimented on should not be less than 50 feet with a head of 5 feet.

121. Rear Apron. The extension of the floor rearward is termed the rear apron. Its statical condition is peculiar, not being subject to any upward hydrostatic pressure as is the case with the fore apron or floor. Inspection of the diagram, Fig. 89, will show that the water pressure acting below the floor is the trapezoid enclosed between the piezometric line and the floor level; whereas the downward pressure is represented by the rectangle H_1A_1HA, which is considerably larger. Theoretically no weight is required in the rear apron, the only proviso being that it must be impervious and have a water-tight connection with the weir wall, otherwise the

incidence of H may fall between the rear apron and the rest of the work, rendering the former useless. Such a case has actually occurred. It is, however, considered that the rear apron must be of a definite weight, as otherwise the percolation of water underneath it would partake of the nature of a surface flow, and so prevent any neutralization of head caused by friction in its passage through sand. Consequently, the effective thickness, or rather $t(\rho-1)$ should not be less than four feet. Its level need not be the same as that of the fore apron or floor. In fact, in some cases it has been constructed level or nearly so with the permanent crest

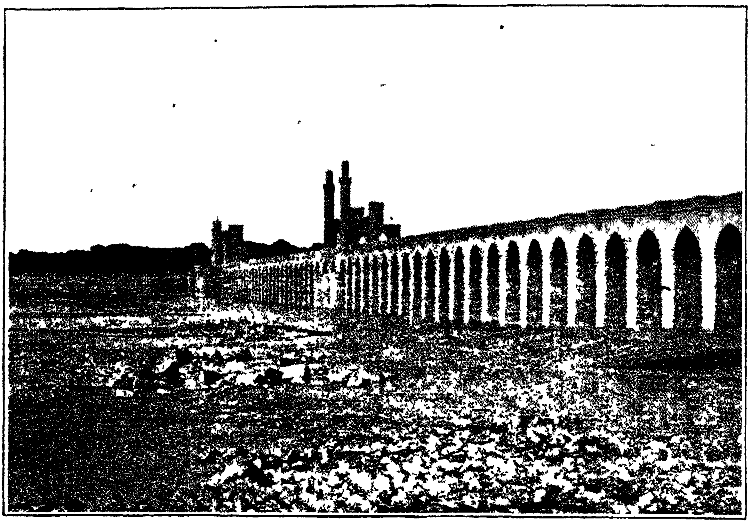


Fig 90 View of Grand Barrage over Nile River

of the drop wall. But this disposition has the effect of reducing the coefficient of discharge over the weir and increasing the afflux or head water level, which is open to objection. The best position is undoubtedly level with the fore apron.

Another point in favor of the rear apron is the fact that it is free from either hydrostatic pressure or the dynamic force of falling water, to which the fore apron is subject; it can, therefore, be constructed of more inexpensive material. Clay consolidated when wet, i.e., puddle, is just as effective in this respect as the richest cement masonry or concrete, provided it is protected from scour where necessary by an overlay of paving or riprap, and has a reli-

able connection with the drop wall and the rest of the work. In old works these properties of the rear apron were not understood, and the stanching of the loose stone rear apron commonly provided, was left to be effected by the natural deposit of silt. This deposit eventually does take place and is of the greatest value in increasing the statical stability of the weir, but the process takes time, and until complete, the work is liable to excess hydrostatic pressure and an insufficient length of enforced percolation, which would allow piping to take place and the foundation to be gradually undermined.

122. First Demonstration of Rear Apron. The value of an impervious rear apron was first demonstrated in the repairs to the Grand Barrage over the Nile, Fig. 90, some time in the eighties. This old work was useless owing to the great leakage that took place whenever the gates were lowered and a head of water applied. In order to check this leakage, instead of driving sheet piling, which

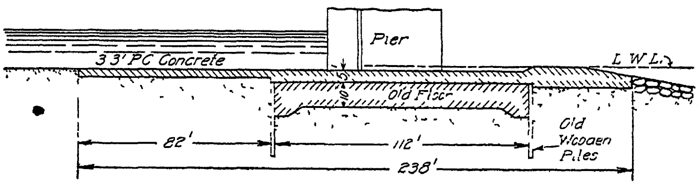


Fig 91. Section Showing Repairs Made on the Grand Barrage

it was feared would shake the foundations, an apron of cement masonry 240 feet wide and 3.28 feet thick, Fig. 91, was constructed over the old floor, extending upstream 82 feet beyond it. This proved completely successful. By means of pipes set in holes drilled in the piers, cement mortar was forced under pressure into all the interstices of the rubble foundations, filling up any hollows that existed, thus completely stanching the foundations. So effectually was the structure repaired that it was rendered capable of holding up about 13 feet of water; whereas, prior to reconstruction, it was unsafe with a head of a little over three feet. The total length of apron is 238 feet, of which 82 feet projects upstream beyond the original floor and 44 feet downstream, below the floor itself, the latter having a width of 112 feet. The head H being 13 feet, and L being 238 feet, c, or the percolation factor is $\frac{238}{13} = 18$, which is the exact value assigned for Nile sand in *Class I*,

section 118. This value was not originally derived from the Grand Barrage, but from another work.

The utility of this barrage has been further augmented by the construction of two subsidiary weirs below it, see Figs. 92 and 106, across the two branches of the Nile delta. These are ten feet high and enable an additional height of ten feet to be held up by the gates of the old barrage, the total height being now $22\frac{1}{4}$ feet. The increased rise in the tail water exactly compensates for the additional head on the work as regards hydrostatic pressure, but the moment of the water pressure on the base of the masonry piers

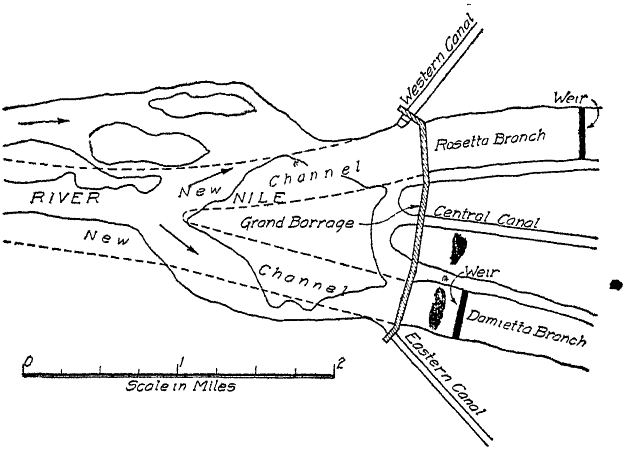


Fig 92 Plan of Grand Barrage over Nile River Showing Also Location of Damietta and Rosetta Weirs

will be largely increased, viz, as from 13^3 to 22^3-10^3, or from 2197 to 9648.

The barrage, which is another word for "open dam" or bulkhead dam, is, however, of very solid and weighty construction, and after the complete renewal of all its weak points is now capable of safely enduring the increased stress put upon the superstructure. We have seen, in section 119, that the width of the impervious fore apron should be $L=3\sqrt{cH}$, formula (**34**). This width of the floor is affected by two considerations, first, the nature of the river bed, which can best be represented by its percolation factor c and second, by the height of the overfall including the crest shutters if any, which will be designated by H_a to distinguish it from H, which represents the difference between head and tail water and also

TABLE I

Showing Actual and Calculated Values of L_1 or Talus Width

Formula (35), $L_1 = 10c\sqrt{\frac{H_b}{10}} \times \sqrt{\frac{q}{75}}$

River	Name of Work	Type	c	H_b	q	Length L_1	
						Calculated	Actual
Ganges	Narora	A	15	10	75	150	140–170
Coleroon	Coleroon	A	12	4½	100	92	72
Vellar	Pelandorai	A	9	11	100	108	101
Tampra-parni	Srivakantham	A	12	6	90	102	106
Chenab	Khanki	B	15	7	150	182	170
Chenab	Merala	B	15	7	150	182	203
Jhelum	Rasul	B	15	6	155	160	135
Penner	Adimapali	B	12	8½	184	172	184
Penner	Nellore	B	12	9	300	228	232
Penner	Sangam	B	12	10	147	168	145
Godaveri	Dauleshwnam	B	12	13	100	158	217
Jumna	Okhla	C	15	10	140	210	210
Kistna	Beswada	C	12	13	223	236	220
Son	Dehri	C	12	8	66	100	96
Mahanadi	Jobra	C	12	100	140	163	143
Madaya	Madaya	C	12	8	280	207	235
Colorado	Laguna	C	15	10	below minimum	140	200

Type A has a direct overfall, with horizontal floor at L. W. L, as in Figs. 91, 93, and 95.

Type B has breast wall followed by a sloping impervious apron, Figs 96 and 97.

Type C has breast wall followed by pervious rock fill, with sloping surface and vertical body walls, Figs. 101 to 106.

from H_b the height of the permanent crest above L. W. L. Taking the Narora weir as standard, a length of floor equal to $3\sqrt{cH}$ $=3\sqrt{15\times 13}=42$ feet, is deemed to be the correct safe width for

a weir 13 feet in height; where the height is more or less, the width should be increased or reduced in proportion to the square root of the height and that of the factor c.

123. Riprap to Protect Apron. Beyond the impervious floor a long continuation of riprap or packed stone pitching is required. The width of this material is clearly independent of that appropriate to the floor, and consequently will be measured from the same starting point as the floor, viz, from the toe of the drop wall. The formula for overfall weirs is

$$L_1 = 10c\sqrt{\frac{H_b}{10}} \times \sqrt{\frac{q}{75}} \qquad (35)$$

For sloping aprons, type B, the coefficient of c will be 11
Then

$$L_1 = 11c\sqrt{\frac{H_b \times q}{10\ \ 75}} \qquad (35a)$$

This formula is founded on the theory that the distance of the toe of the talus from the overfall will vary with the square root of the height of the obstruction above low water, designated by H_b, with the square root of the unit flood discharge over the weir crest q, and directly with c, the percolation factor of the river sand. The standard being these values; viz, 10, 75, and 10, respectively, in Narora weir. This height, H_b is equal to H when there are no crest shutters, and is always the depth of L. W. L. below the permanent masonry crest of the weir. This formula, though more or less empirical, gives results remarkably in consonance with actual value, and will, it is believed, form a valuable guide to design. Table I will conclusively prove this. As nearly all the weirs of this class have been constructed in India, works in that country are quoted as examples.

124. Example of Design Type A. Another example of design in type A will now be given, Fig. 93, the dimensions being those of an actual work, viz, the Narora weir over the Ganges River, the design being thus an alternative for that work, the existing section of which is shown in Fig. 95 and discussed in section 125. The data on which the design is based, is as follows: sand, class 2; percolation factor $c = 15$; H or difference between head and low water, the latter being always symbolized by L. W. L., 13 feet, unit dis-

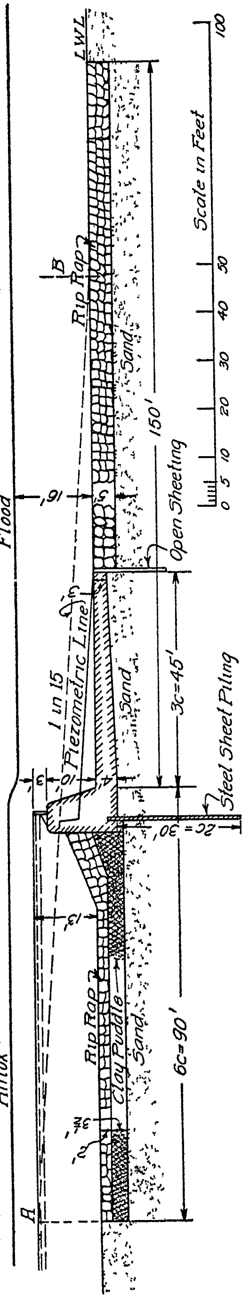


Fig 93 Design of Weir on Sand Having Direct Overfall and Horizontal Flooring

charge over weir $q=75$ second-feet, the total length of the impervious apron and vertical obstructions will, therefore, have to be $L=cH=15\times13=195$ feet.

The first point to be determined is the length of the floor or fore apron. Having fixed this length, the balance of L will have to be divided among the rear apron and the vertical sheet piling. It is essential that this minimum length be not exceeded, as it is clearly of advantage to put as much of the length into the rear apron as possible, owing to the inexpensive nature of the material of which it can be constructed. According to formula (2), $L=42$ feet, which is nearly equivalent to $3c$, or 45 feet, there thus remains $10c$ to be proportioned between the rear apron and the vertical curtain. If the latter be given a depth of $2c$, or 30 feet the length of travel down and up will absorb $4c$, leaving $6c$, or 90 feet for the rear apron. The measurement is taken from the toe of the drop wall. The neutralization of the whole head of 13 feet is thus accomplished. A second curtain will generally be desirable at the extremity of the fore apron as a precautionary measure to form a protection in case the loose riprap downstream from the apron is washed out or sinks. This curtain must have open joints to offer as little obstruction to percolation as possible. The outline of the pressure area, that is the piezometric line, is drawn as follows: $cH=195$ feet is measured horizontally on the base line of the pressure area, that is, at L. W. L. from a

Fig 94 View of Narora Weir and Headworks

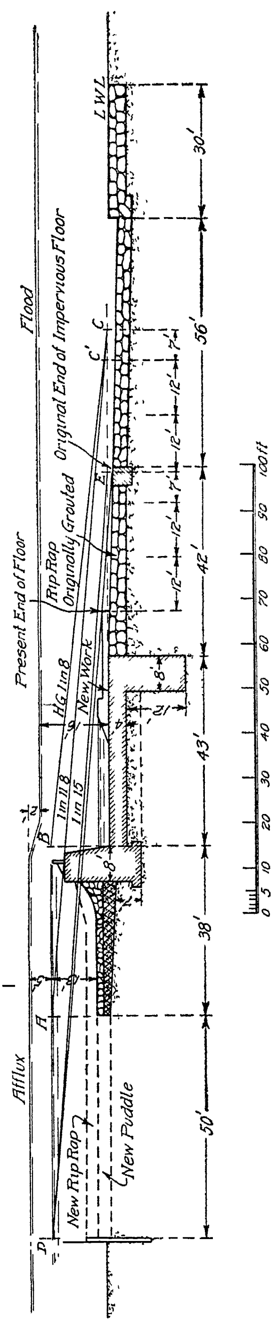


Fig 95 Diagram of Narora Weir Showing Changes in Profile after Failure of Dam
The floor of this weir blew up due to uplift Final repairs have proved satisfactory

line through A to B. The point B is then joined with A on the head water level at the commencement of the rear apron. The hydraulic gradient will thus be 1 in 15. The intersection of this line BA with a vertical drawn through the first line of curtain is the location of a step of two feet equal to the head absorbed in the vertical travel at this point. Another line parallel to the hydraulic gradient is now drawn to the termination of the fore apron, this completes the piezometric line or the upper outline of the pressure area.

With regard to the floor thickness at the toe of the drop wall, the value of h, or loss of head due to percolation under the rear apron, is 6 feet, from the rear curtain, 4 feet; total 10 feet. $H-h$ is, therefore, $13-10=3$. The thickness of the floor according to formula (**33**) where $(H-h)=3$ feet comes to $\frac{4}{3}\times 3=4$ feet, the value of ρ being assumed at 2. The floor naturally tapers toward its end where the uplift is nil. The thickness at this point is made 3 feet which is about the minimum limit. There remains now the talus of riprap, its length from formula (**35**) is

$$L=10c\sqrt{\frac{H^b}{10}}\sqrt{\frac{q}{75}}=150 \text{ feet}=10c$$

The thickness of the talus is generally four—often five feet—and is a matter of judgment considering the nature of the material used.

125. Discussion of Narora Weir. The Narora weir itself, Fig. 94, forms

a most instructive object lesson, demonstrating what is the least correct base width, or length of percolation consistent with absolute safety, that can be adopted for sands of class 2. The system of analyzing graphically an existing work with regard to hydraulic gradient is exemplified in Fig. 95 under three separate conditions; *first,* as the work originally stood, with a hydraulic gradient of 1 in 11.8; *second,* at the time of failure, when the floor and the grouted riprap blew up. On this occasion owing to the rear apron having been washed away by a flood the hydraulic grade fell to 1 in 8; *third,* after the extension of the rear apron and curtailment of the fore apron had been effected. Under the first conditions the horizontal component of the length of travel or percolation L from A to E is 123 feet. The total length is made up of three parts: First, a step down and up in the foundation of the drop wall of 7 feet; second, a drop down and up of 12 feet either side of the downstream curtain wall; third, the horizontal distance 123 as above. The rise at the end of the floor is neglected. The total value of L is then $123+7+12+12=154$. This is set out on a horizontal line to the point C. AC is then the hydraulic gradient.

This demonstrates that the hydraulic gradient was originally something under 1 in 12, and in addition to this the floor is very deficient in thickness. The hydrostatic pressure on the floor at the toe of the drop wall is 8 feet. To meet this the floor has a value of $t\rho$ of only 5 feet. The specific gravity of the floor will not exceed 2, as it was mostly formed of broken brick concrete in hydraulic mortar. The value of $\rho-1$ will, therefore, be unity, the floor being submerged. In spite of this, the work stood intact to all external appearance for twenty years, when a heavy freshet in the river set up a cross current which washed out that portion of the rear apron nearest the drop wall, thus rendering the rest useless, the connection having been severed.

On this occurrence, failure at once took place, as the floor had doubtless been on the point of yielding for some time. In fact, this state of affairs had been suspected, as holes bored in the floor very shortly before the actual catastrophe took place showed that a large space existed below it, full, not of sand, but of water. Thus the floor was actually held up by the hydrostatic pressure; otherwise it must have collapsed. The removal of the rear apron caused this

pressure to be so much increased that the whole floor, together with the grouted pitching below the curtain, blew up.

The hydraulic gradient BC is that at the time of the collapse. It will be seen that it is now reduced to 1 in 8. The piezometric line is not shown on the diagram.

In restoring the work the rear apron was extended upstream as shown dotted in Fig. 95, to a distance of 80 feet beyond the drop wall, and was made five feet thick. It was composed of puddle covered with riprap and at its junction with the drop wall was provided with a solid masonry covering. The puddle foundation also was sloped down to the level of the floor base to form a ground connection with the drop wall. At its upstream termination sheet piling was driven to a depth of twelve feet below floor level.

The grouted pitching in the fore apron was relaid dry, except for the first ten feet which was rebuilt in mortar, to form a continuation of the impervious floor. Omitting the mortar has the effect of reducing the pressure on the floor. Even then the uplift would have been too great, so a water cushion 2 feet deep was formed over the floor by building a dwarf wall of concrete (shown on the section) right along its edge. This adds 1 foot to the effective value of t_ρ. It will be seen that the hydraulic gradient now works out to 1 in 15. A value for c of 15 has been adopted for similar light sands from which that of other sands, as *Classes I* and *III*, have been deduced.

It will be noticed that the crest of this weir is furnished with shutters which are collapsible when overtopped and are raised by hand or by a traveling crab that moves along the crest, raising the shutters as it proceeds. The shutters are 3 feet deep and some 20 feet long. They are held up against the water by tension rods hinged to the weir, and at about $\frac{1}{3}$ the height of the shutter, i.e., at the center of pressure.

126. Sloping Apron Weirs, Type B. Another type of weir, designated B, will now be discussed, in which there is no direct vertical drop, the fore apron not being horizontal but sloping from the crest to the L. W. L. or to a little above it, the talus beyond being also on a flat slope or horizontal.

In the modern examples of this type which will be examined, the height of the permanent masonry weir wall is greatly reduced, with the object of offering as little obstruction as possible to the

passage of flood water. The canal level is maintained by means of deep crest shutters. In the Khanki weir, Fig. 96, the weir proper, or rather bar wall, is 7 feet high above L. W. L., while the shutters are 6 feet high. It, therefore, holds up 13 feet of water, the same as was the case with the Narora weir.

The object of adopting the sloping apron is to avoid construction in wet foundations, as most of it can be built quite in the dry above L. W. L. The disadvantage of this type lies in the constriction of the waterway below the breast wall, which causes the velocity of overfall to be continued well past the crest. With a direct overfall, on the other hand, a depth of 7 feet for water to churn in would be available at this point. This would check the flow and the increased area of the waterway rendered available

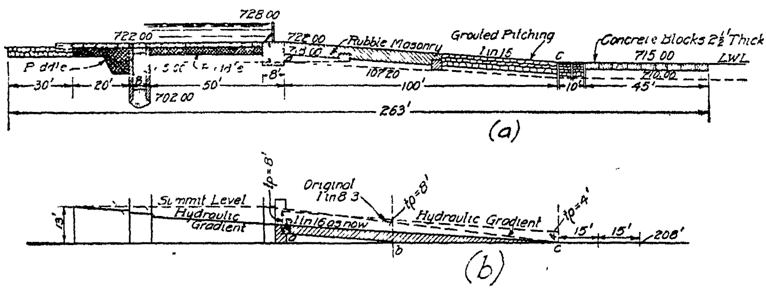


Fig. 96. Profile of Khanki Weir, Showing Restoration Work Similar to that of Narora Weir

should reduce the velocity. For this reason, although the action on the apron is possibly less, that on the talus and river bed beyond must be greater than in the drop wall of type A.

This work, like the former, failed for want of sufficient effective base length, and it consequently forms a valuable object lesson.

As originally designed, no rear apron whatever, excepting a small heap of stone behind the breast wall, was provided. The value of L up to the termination of the grouted pitching is but 108 feet; whereas it should have been cH or $15 \times 13 = 195$ feet. The hydraulic gradient, as shown in Fig. 96b, is only 1 in 8.3. This neglects the small vertical component at the breast wall. In spite of this deficiency in effective base width, the floor, owing to good workmanship, did not give way for some years, until gradually increased piping beneath the base caused its collapse.

Owing to the raised position of the apron, it is not subject to high hydrostatic pressure. At its commencement it is ten feet below the summit level and nine feet of water acts at this point. This is met by four feet of masonry unsubmerged, of s.g. 2, which almost balances it. Thus the apron did not blow up, as was the case with the Narora weir, but collapsed.

Some explanation of the graphical pressure diagram is required, as it offers some peculiarities, differing from the last examples. The full head, or H, is 13 feet. Owing, however, to the raised and sloping position of the apron, the base line of the pressure area will not be horizontal and so coincide with the L. W. L., but will be an inclined line from the commencement a to the point b, where the sloping base coincides with the L. W. L. From b where L. W. L. is reached onward, the base will be horizontal. With a sloping apron the pressure is nearly uniform, the water-pressure area is not wedge shaped but approximates to a rectangle. The apron, therefore, is also properly rectangular in profile, whereas in the overfall type the profile is, or should be, that of a truncated wedge.

127. Restoration of Khanki Weir. After the failure of this work the restoration was on very similar lines to that of the Narora weir. An impervious rear apron, seventy feet long, was constructed of puddle covered with concrete slabs, grouted in the joints. A rear curtain wall consisting of a line of rectangular undersunk blocks twenty feet deep, was provided. These additions have the effect of reducing the gradient to 1 in 16. The masonry curtain having regard to its great cost is of doubtful utility. A further prolongation of the rear apron or else a line of sheet piling would, it is deemed, have been equally effective. Reinforced-concrete sheet piling is very suitable for curtain walls in sand and is bound to supplant the ponderous and expensive block curtain walls which form so marked a feature in Indian works.

128. Merala Weir. Another weir on the same principle and quite recently constructed is the Merala weir at the head of the same historic river, the Chenab, known as the "Hydaspes" at the time of Alexander the Great.

This weir, a section of which is given in Fig. 97, is located in the upper reaches of the river and is subjected to very violent floods; consequently its construction has to be abnormally strong to resist

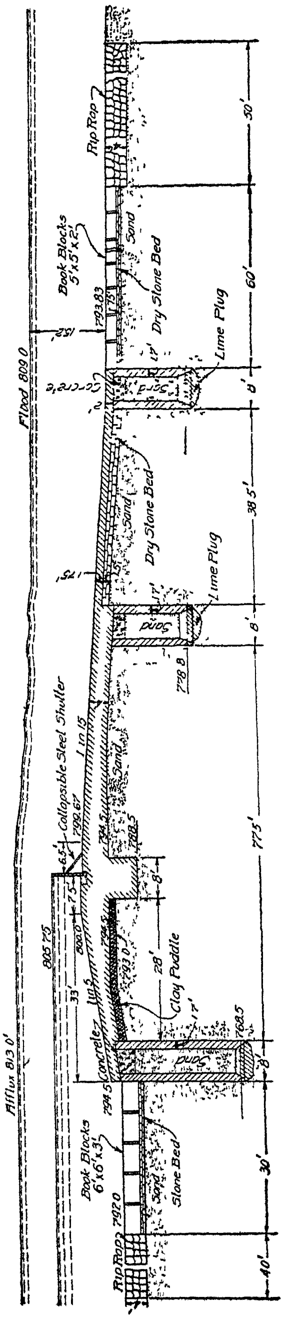


Fig. 97. Section of Merala Weir

the dynamic action of the water. This is entirely a matter of judgment and no definite rules can possibly be given which would apply to different conditions. From a hydrostatic point of view the two lower lines of curtain blocks are decidedly detrimental and could well be cut out. If this were done the horizontal length of travel or percolation will come to 140. The head is 12 or 13 feet. If the latter, c having the value 15 as in the Khanki weir, the value of L will be $15 \times 13 = 195$ feet. The horizontal length of travel is 140 feet and the wanting 55 feet will be just made up by the rear curtain. The superfluity of the two fore lines is thus apparent with regard to hydrostatic requirements. The long impervious sloping apron is a necessity to prevent erosion.

It is a question whether a line of steel interlocking sheet piling is equally efficient as a curtain formed of wells of brickwork 12×8 feet undersunk and connected with piling and concrete filling. The latter has the advantage of solidity and weight lacking in the former. The system of curtain walls of undersunk blocks is peculiar to India. In the Hindia Barrage, in Mesopotamia, Fig. 115, interlocking sheet piling has been largely employed in places where well foundations would have been used in India. This change is probably due to the want of skilled well sinkers, who in India are extremely expert and form a special caste.

The rear apron, in the Merala weir is of as solid construction as the fore apron and is built on a slope right up to crest level; this arrangement facilitates discharge. The velocity of approach must be very great to necessitate huge book blocks of concrete $6 \times 6 \times 3$ feet being laid behind the slope and beyond that a 40-foot length of riprap. The fore apron extends for 93 feet beyond the crest, twice as long as would be necessary with a weir of type A under normal conditions. The distance L of the talus is 203 feet against 182 feet calculated from formula (**35a**). That of the lower weir at Khanki is 170 feet. This shows that the empirical formula gives a fair approximation.

The fore apron in type B will extend to the toe of the slope or glacis. It is quite evident that the erosive action on a sloping apron of type B is far greater than that on the horizontal floor of type A; the

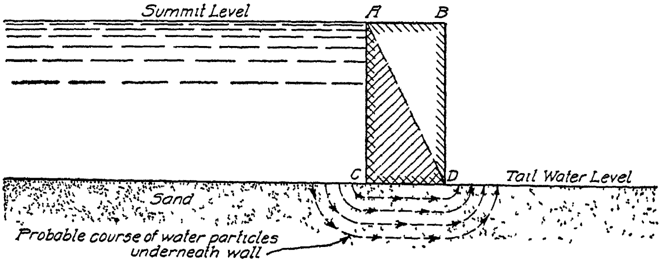


Fig. 98. Diagram Showing Effect of Percolation under a Wall Built on Sand

uplift however is less, consequently the sloping apron can be made thinner and the saving thus effected put into additional length.

129. Porous Fore Aprons. The next type of weir to be dealt with is type C. As it involves some fresh points, an investigation of it and the principles involved will be necessary. The previous examples of types A and B have been cases where the weir has as appendage an impervious fore apron which is subject to hydrostatic pressure. There is another very common type which will be termed C, in which there is no impervious apron and the material which composes the body of the weir is not solid masonry but a porous mass of loose stone the only impervious parts being narrow vertical walls. In spite of this apparent contrariety it will be found that the same principle, viz, that of length of enforced percolation, influences the design in this type as in the others.

Fig. 98 represents a wall upholding water to its crest and resting on a pervious substratum, as sand, gravel, or boulders, or a mixture of all three materials. The hydraulic gradient is *AD;* the upward pressure area *ACD*, and the base *CD* is the travel of the percolation. Unless this base length is equal to *AC* multiplied by the percolation factor obtained by experiment, piping will set in and the wall be undermined. Now as shown in Fig. 99, let a mass of loose stone be deposited below the wall. The weight of this stone will evidently have an appreciable effect in checking the disintegration and removal of the sand foundation. The water will not have a free untrammeled egress at *D;* it will, on the contrary, be compelled to rise in the interstices of the mass to a certain height *EE* determined by the extent to which the loose stones cause obstruction to the flow.

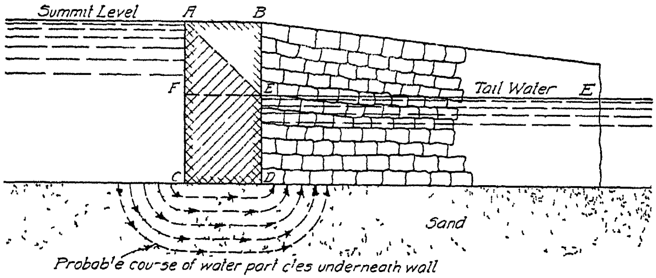


Fig 99 Effect on Percolation Due to Stones below Weir Wall of Fig 98

The resulting hydraulic gradient will now be *AE*—flatter than *AD*, but still too steep for permanency.

In Fig. 100, the wall is shown backed by a rear apron of loose stone, and the fore apron extended to *F*. The water has now to filter through the rear apron underneath the wall and up through the stone filling in the fore apron. During this process a certain amount of sand will be washed up into the porous body and the loose stone will sink until the combined stone and sand forms a compact mass, offering a greater obstruction to the passage of the percolating water than exists in the sand itself and possessing far greater resisting power to disintegration. This will cause the level of water at *E* to rise until equilibrium results. When this is the case the hydraulic gradient is flattened to some point near *F*. If a sufficiently long body is provided, the resulting gradient will be equal to that found by experiment to produce permanent equilibrium.

The mass after the sinking process has been finished is then made good up to the original profile by fresh rock filling. At F near the toe of the slope the stone offers but little resistance either by its weight or depth; so it is evident that the slope of the prism should be flatter than the hydraulic gradient.

The same action takes place with the rear apron, which soon becomes so filled with silt, as to be impervious or nearly so to the passage of water. But unless silt is deposited in the river bed behind, as eventually occurs right up to crest level, the thin portion of the rear slope, as well as the similar portion of the fore slope, cannot be counted as effective. Consequently out of the whole base length this part GF, roughly, about one-quarter, can be deemed inefficient as regards length of enforced percolation. As the consolidated lower part of the body of the weir gains in consistency, it can well be subject to hydrostatic pressure. Consequently, the value of $t\rho$ of the mass should be in excess of that of $H-h$, just as was the case with an impervious floor.

130. Porous Fore Aprons Divided by Core Walls. In Fig. 101 a still further development is effected by the introduction of vertical body or core walls of masonry in the pervious mass of the fore apron. These impervious obstructions materially assist the stability of the foundation, so much so that the process of underscour and settlement which must precede the balancing of the opposing forces in the purely loose stone mass need not occur at all, or to nothing like the same extent. If the party walls are properly spaced, the surface slope can be that of the hydraulic gradient itself and thus ensure equilibrium. This is clearly illustrated in the diagram. The water passing underneath the wall base CD will rise to

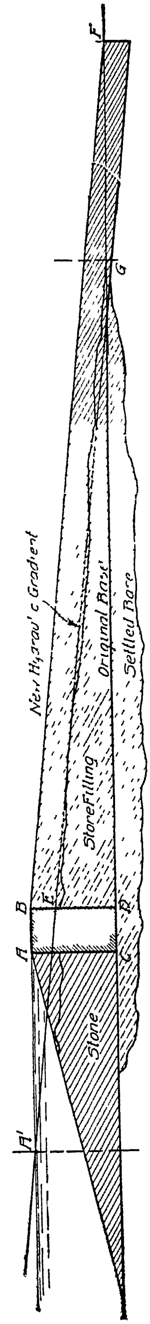


Fig 100. Effect on Percolation When Weir Wall Is Provided with Rear Apron of Loose Stone and Extensive Fore Apron of Stone Filling

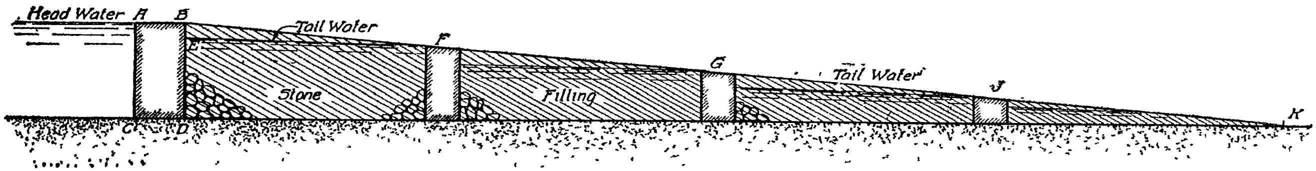


Fig 101. Diagram of Weir Showing Porous Fore Apron Divided by Core Walls

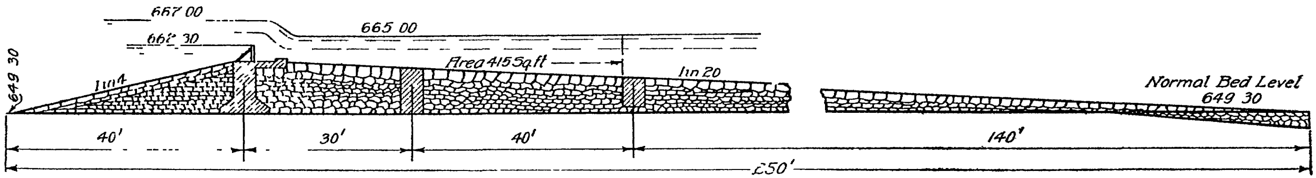


Fig 102 Okhla Rock-Fill Weir

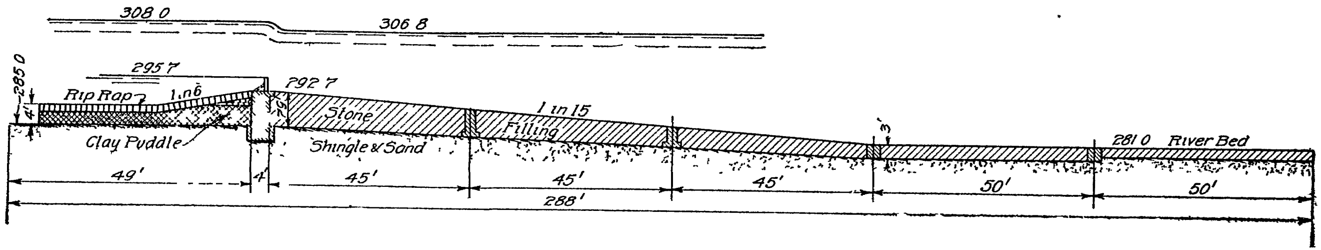


Fig 103 Madaya Rock-Fill Weir

the level F, the point E being somewhat higher; similar percolation under the other walls in the substratum will fill all the partitions full of water. The head AC will, therefore, be split up into four steps.

Value of Rear Apron Very Great. The value of water tightness in the rear apron is so marked that it should be rendered impervious by a thick under layer of clay, and not left entirely to more or less imperfect surface silt stanching, except possibly in the case of high dams where a still settling pool is formed in rear of the work.

131. Okhla and Madaya Weirs. In Fig. 102 is shown a detailed section of the Okhla rock-filled weir over the Jumna River, India. It is remarkable as being the first rock-filled weir not provided with any lines of curtain walls projecting below the base line, which has hitherto invariably been adopted. The stability of its sand foundation is consequently entirely dependent on its weight and its effective base length. As will be seen, the section is provided with two body walls in addition to the breast wall. The slope of the fore apron is 1 in 20. It is believed that a slope of 1 in 15 would be equally effective, a horizontal talus making up the continuation, as has been done in the Madaya weir, Fig. 103, which is a similar work but under much greater stress.

The head of the water in the Okhla weir is 13 feet, with shutters up and weir body empty of water—a condition that could hardly exist. This would require an effective base length, L, of 195 feet; the actual is 250 feet. But, as noted previously, the end parts of the slopes cannot be included as effective; consequently the hydraulic gradient will not be far from 1 in 15. The weight of the stone, or $t\rho$, exactly balances this head at the beginning, as it is $10 \times 1.3 = 13$ feet. If the water were at crest level and the weir full of water, $t\rho$ would equal 8 feet, or rather a trifle less, owing to the lower level of the crest of the body wall. The head of 13 feet is broken up into four steps. The first is 3 feet deep, acting on a part of the rear apron together with 30 feet of the fore apron, say, 1 in 15; the rest are 1 in 20. A slope of 1 in 15 for the first party wall would cut the base at a point 40 feet short of the toe. Theoretically a fourth party wall is required at this point, but practically the riprap below the third dwarf wall is so stanched with sand as not to afford a free egress for the percolation; consequently the slope may

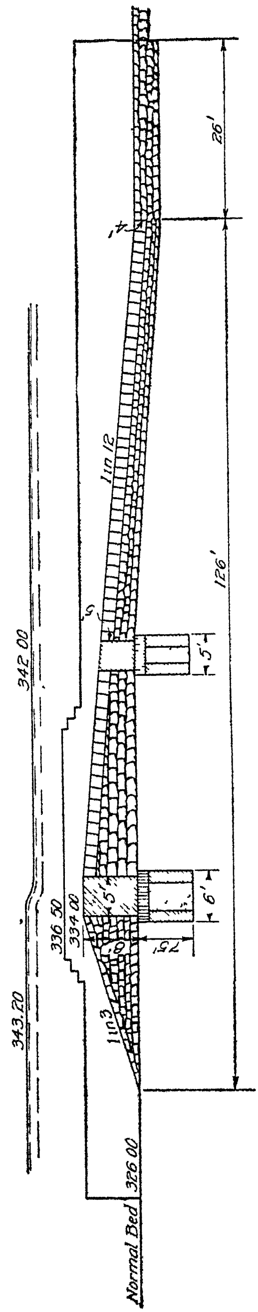


Fig 104 Section of Dehri Weir Showing Steeper Gradient Than Fig 103

be assumed to continue on to its intersection with the horizontal base. As already noted, material would be saved in the section by adopting a reliably stanch rear apron and reducing the fore slope to 1 in 15, with a horizontal continuation as was done in the Madaya weir.

Economy of Type C. This type C is only economical where stone is abundant. It requires little labor or masonry work. On the other hand, the mass of the material used is very great, much greater, in fact, than is shown by the section. This is owing to the constant sinking and renewal of the talus which goes on for many years after the first construction of the weir.

The action of the flood on the talus is undoubtedly accentuated by the contraction of the waterway due to the high sloping apron. The flood velocity 20 feet below the crest has been gaged as high as 18 feet per second. This would be very materially reduced if the A type of overfall were adopted, as the area of waterway at this point would be more than doubled.

132. Dehri Weir. Another typical example of this class is the Dehri weir over the Son River, Fig. 104. The value of L, if the apexes of the two triangles of stone filling are deducted and the curtain walls included, comes to about $12\,H$, 12 being adopted for this class of coarse sand. The curtain walls, each over 12,500 feet long, must have been enormously costly. From the experience of Okhla,

a contemporary work, on a much worse class of sand, curtain protection is quite unnecessary if sufficient horizontal base width is provided. The head on this weir is 10 feet, and the height of breast wall 8 feet, *tp* is, therefore, 1.3×8=10.1, which is sufficient, considering that the full head will not act here. The lines of curtains could be safely dispensed with if the following alterations were made: (1) Rear apron to be reliably stanched in order to throw back the incidence of pressure and increase the effective base length; (2) three more body walls to be introduced; (3) slope 1 in 12 retained, but base to be dredged out toward apex to admit of no thickness under five feet. This probably would not cause any increase in the quantities of masonry above what they now are, and would entirely obviate the construction of nearly five miles of undersunk curtain blocks.

133. Laguna Weir. The Laguna weir over the Colorado River, the only example of type C in the United States is shown in Fig. 105. Compared with other examples it might be considered as somewhat too wide if regard is had to its low unit flood discharge, but the inferior quality of the sand of this river probably renders this necessary. The body walls are undoubtedly not sufficiently numerous to be properly effective. The provision of an impervious rear apron would also be advantageous.

134. Damietta and Rosetta Weirs. The location of the Damietta and Rosetta subsidiary weirs, Fig. 106, which have been rather recently erected below the old Nile barrage, is shown in Fig. 92. These weirs are of type C, but the method of construction is quite

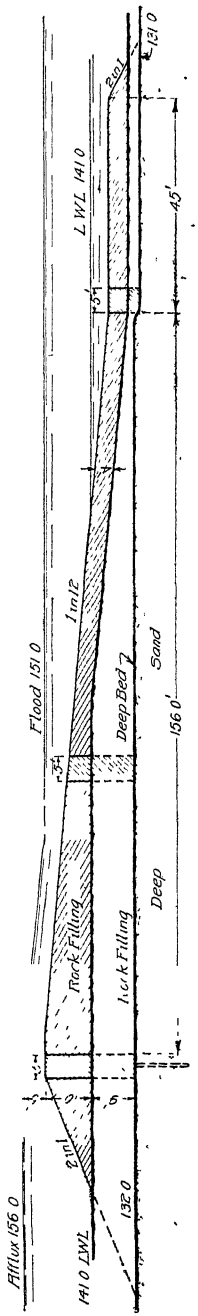


Fig 105 Section of Laguna Weir over Colorado River

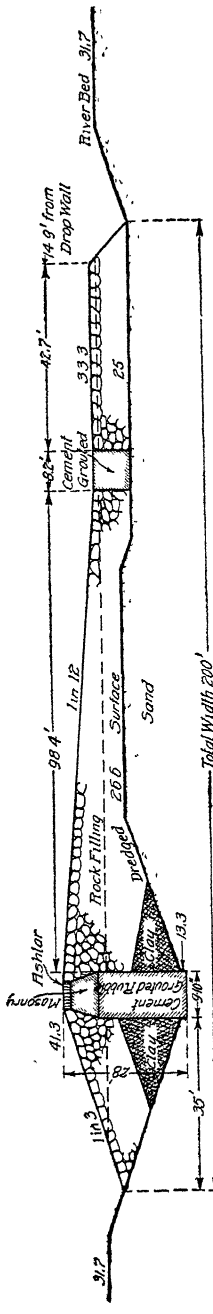


Fig 106 Profile of Dametta and Rossetta Weirs

novel and it is this alone that renders this work a valuable object lesson. The deep foundation of the breast wall was built without any pumping, all material having been deposited in the water of the Nile River. First the profile of the base was dredged out, as shown in the section. Then the core wall was constructed by first depositing, in a temporary box or enclosure secured by a few piles, loose stone from barges floated alongside. The whole was then grouted with cement grout, poured through pipes let into the mass. On the completion of one section all the appliances were moved forward and another section built, and so on until the whole wall was completed. Clay was deposited at the base of the core wall and the profile then made up by loose stone filling.

This novel system of subaqueous construction has proved so satisfactory that in many cases it is bound to supersede older methods. Notwithstanding these innovations in methods of rapid construction, the profile of the weir itself is open to the objection of being extravagantly bulky even for the type adopted, the base having been dredged out so deep as to greatly increase the mean depth of the stone filling.

It is open to question whether a row or two rows of concrete sheet piles would not have been just as efficacious as the deep breast wall, and would certainly have been much less costly. The pure cement grouting was naturally expensive, but the admixture of sand proved unsatisfactory as the two materials of different specific gravity separated and formed layers; consequently,

pure cement had to be used. It may be noted that the value of L here is much less than would be expected. At Narora weir it is $11c$, or 165 feet. Here, with a value of c of 18, it is but $8\frac{1}{2}c$, or 150 feet, instead of 200 feet, according to the formula. This is due to the low flood velocity of the Nile River compared with the Ganges.

135. The Paradox of a Pervious Dam. From the conditions prevailing in type C it is clear that an impervious apron as used in types A and B is not absolutely essential in order to secure a safe length of travel for the percolating subcurrent. If the water is free to rise through the riprap and at the same time the sand in the river bed is prevented from rising with it, the practical effect is the same as with an impervious apron. "Fountaining", as spouting sand is technically termed, is prevented and consequently also "piping". This latter term defines the gradual removal of sand from beneath a foundation by the action of the percolating under current. Thus the apparent paradox that a length of filter bed, although pervious, is as effective as a masonry apron would be. The hydraulic gradient in such case will be steeper than allowable under the latter circumstance. Filter beds are usually composed of a thick layer of gravel and stone laid on the sand of the river bed, the small stuff at the bottom and the larger material at the top. The ideal type of filter is one composed of stone arranged in sizes as above stated of a depth of 4 or 5 feet covered with heavy slabs or book blocks of concrete; these are set with narrow open intervals between blocks as shown in Figs. 96 and 97. Protection is thus afforded not only against scour from above but also from uplift underneath. Although the subcurrent of water can escape through a filter its free exit is hindered, consequently some hydrostatic pressure must still exist below the base, how much it is a difficult matter to determine, and it will therefore be left out of consideration. If the filter bed is properly constructed its length should be included in that of L or the length of travel. Ordinary riprap, unless exceptionally deep, is not of much, if any, value in this respect. The Hindia Barrage in Mesopotamia, Fig. 115, section 145, is provided with a filter bed consisting of a thick layer of stone 65.5 feet wide which occurs in the middle of the floor. The object of this is to allow the escape of the subcurrent and reduce the uplift on the dam and on that part of the floor which is impervious.

136. Crest Shutters. Nearly all submerged river weirs are provided with crest shutters 3 to 6 feet deep, 6 feet being the height adopted in the more recent works. These are generally raised by means of a traveling crane running on rails just behind the hinge of the gate. When the shutters are tripped they fall over this railway. In the case of the Merala weir, Fig. 97, the raising of the shutters is effected from a trolley running on overhead wires strung over steel towers erected on each pier. These piers or groins are 500 feet apart. The 6-foot shutters are 3 feet wide, held up by hinged struts which catch on to a bolt and are easily released by hand or by chains worked from the piers. On the Betwa weir the shutters, also 6 feet deep, are automatic in action, being hinged to a tension rod at about the center of pressure, consequently when overtopped they turn over and fall. Not all are hinged at the same height; they should not fall simultaneously but ease the flood gradually. The advantage of deep shutters is very great as the permanent weir can be built much lower than otherwise would be necessary, and thus offer much less obstruction to the flood. The only drawback is that crest shutters require a resident staff of experienced men to deal with them.

The Laguna weir, Fig. 105, has no shutters. The unit flood discharge of the Colorado is, however, small compared with that of the Indian rivers, being only 22 second-feet, whereas the Merala weir discharges 150 second-feet per foot run of weir, consequently shutters in the former case are unnecessary.

OPEN DAMS OR BARRAGES

137. Barrage Defined. The term "open dam", or barrage, generally designates what is in fact a regulating bridge built across a river channel, and furnished with gates which close the spans as required. They are partial regulators, the closure being only effected during low water. When the river is in flood, the gates are opened and free passage is afforded for flood water to pass, the floor being level with the river bed. Weir scouring sluices, which are indispensable adjuncts to weirs built over sandy rivers, belong practically to the same category as open dams, as they are also partial regulators, the difference being that they span only a portion of the river instead of the whole, and further are subject to great

scouring action from the fact that when the river water is artificially raised above its normal level by the weir, the downstream channel is empty or nearly so.

Function of Weir Sluices. The function of weir sluices is two-fold: First, to train the deep channel of the river, the natural course of which is obliterated by the weir, past the canal head, and to retain it in this position. Otherwise, in a wide river the low water channel might take a course parallel to the weir crest itself, or else one distant from the canal head, and thus cause the approach channel to become blocked with deposit.

Second, by manipulating the sluice gates, silt is allowed to deposit in the slack water in the deep channel. The canal is thereby preserved from silting up, and when the accumulation becomes excessive, it can be scoured out by opening the gates.

The sill of the weir sluice is placed as low as can conveniently be managed, being generally either at L. W. L. itself, or somewhat higher, its level generally corresponding with the base of the drop or breast wall. Thus the maximum statical head to which the work is subjected is the height of the weir crest plus that of the weir shutters, or H_1.

The ventage provided is regulated by the low-water discharge of the river, and should be capable of taking more than the average dry season discharge. In one case, that of the Laguna weir, where the river low supply is deficient, the weir sluices are designed to take the whole ordinary discharge of the river excepting the highest floods. This is with the object of maintaining a wide, deep channel which may be drawn upon as a reservoir. This case is, however, exceptional.

As the object of a weir sluice is to pass water at a high velocity in order to scour out deposit for some distance to the rear of the work, it is evident that the openings should be wide, with as few obstructions as possible in the way of piers, and should be open at the surface, the arches and platform being built clear of the flood level. Further, in order to take full advantage of the scouring power of the current, which is at a maximum at the sluice itself, diminishing in velocity with the distance to the rear of the work, it is absolutely necessary not only to place the canal head as close as possible to the weir sluices, but to recess the head as little

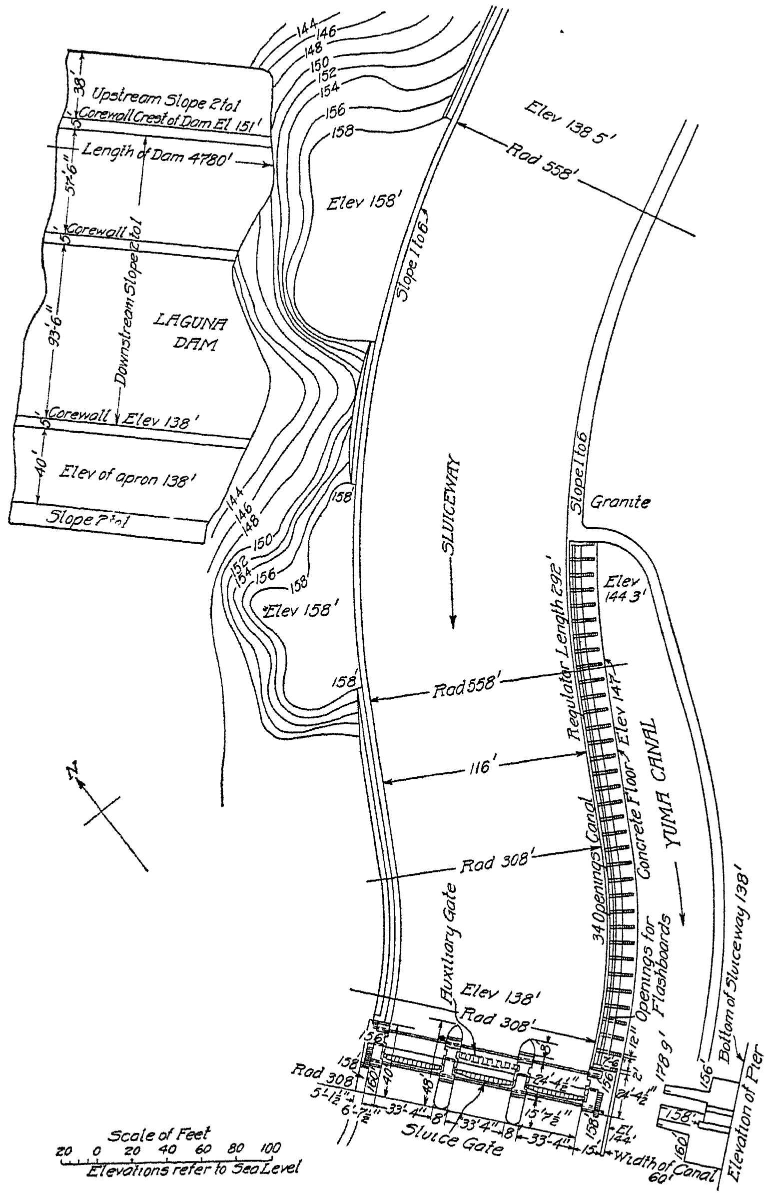


Fig. 107. Plan of Laguna Weir-Scouring Sluices

as practicable behind the face line of the abutment of the end sluice vent.

With regard to canal head regulators or intakes, the regulation effected by these is entire, not partial, so that these works are subjected to a much greater statical stress than weir sluices, and consequently, for convenience of manipulation, are usually designed with narrower openings than are necessary or desirable in the latter. The design of these works is, however, outside the scope of the subject in hand.

138. Example of Weir Scouring Sluice. Fig. 107 is an excellent example of a weir scouring sluice, that attached to the Laguna

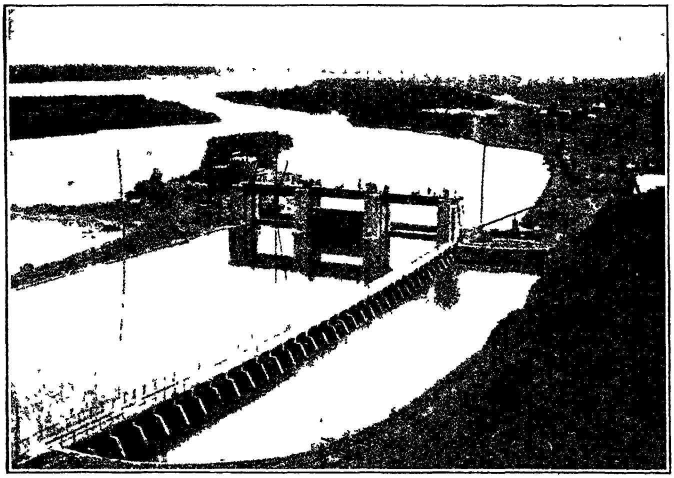


Fig 108. View of Yuma Canal and Sluiceway Showing Sluice Gates under Construction

weir, the profile of which was given in Fig. 105. The Yuma canal intake is placed clear of the sandy bed of the river on a rock foundation and the sluiceway in front of it is also cut through solid rock independent of the weir. At the end of this sluiceway and just past the intake the weir sluices are located, consisting of three spans of $33\frac{1}{2}$ feet closed by steel counterweighted roller gates which can be hoisted clear of the flood by electrically operated winches. The gates are clearly shown in Fig. 108, which is from a photograph taken during the progress of the work. The bed of the sluiceway is at *El.* 138.0, that of the canal intake sill is 147.0, and that of the

weir crest 151.0 hence the whole sluiceway can be allowed to fill up with deposit to a depth of 9 feet, without interfering with the

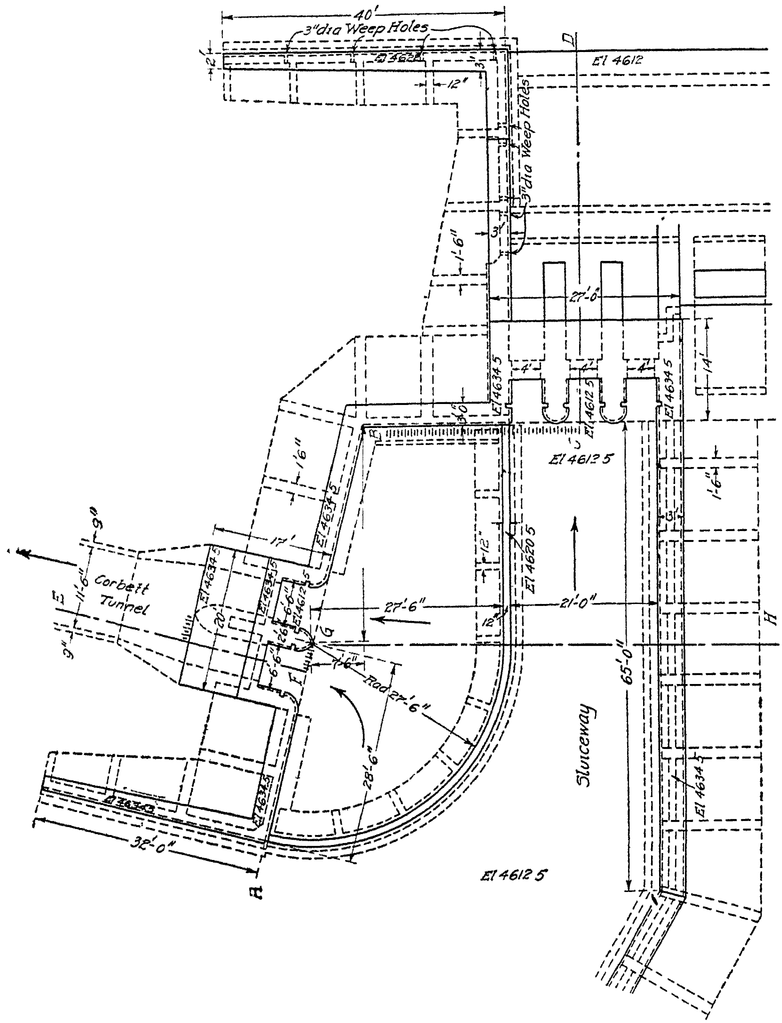


Fig 109 Plan of Weir Sluices for Corbett Dam on Shoshone River, Wyoming

discharge of the canal, or if the flashboards of the intake are lowered the sluiceway can be filled up to *El.* 156 which is the level of the top of the draw gates, i.e., 18 feet deep. The difference between high

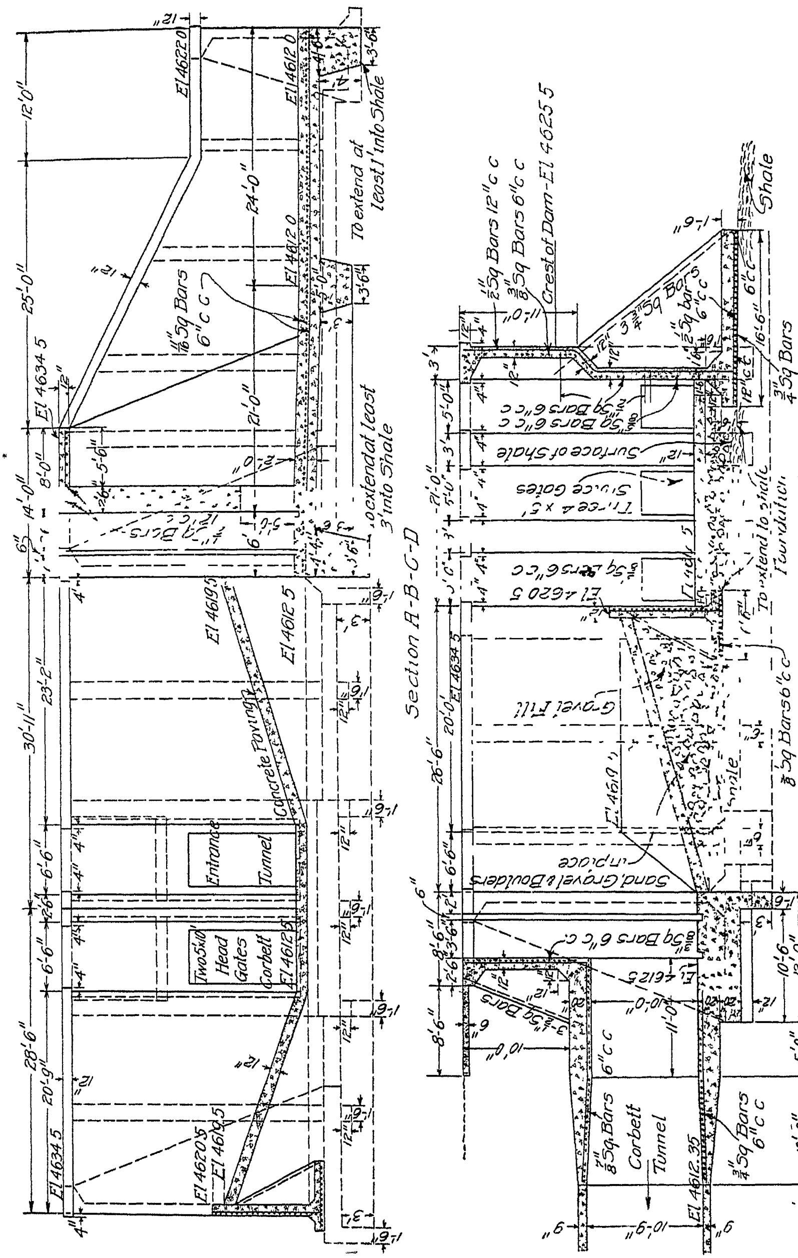


Fig 110. De-ign Diagrams for Wen Sluices of Corbett Dam

Fig 111. View of Corbett Dam on Shoshone River in Winter

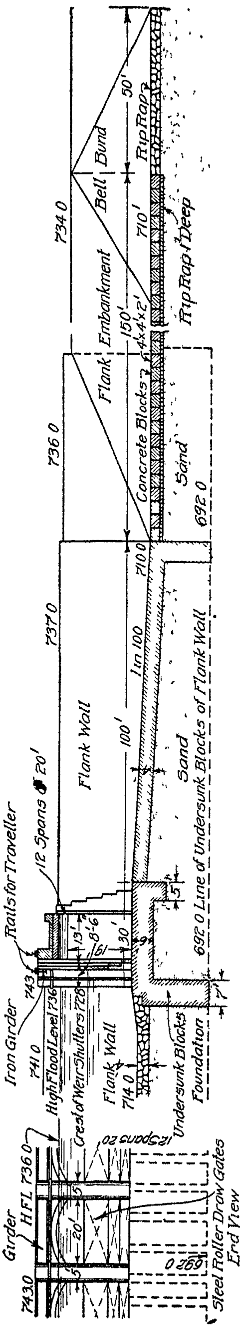


Fig. 112 Section of Undersluices of Khanki Weir over Chenab River, India

water above and below the sluice gates is 11 feet, consequently when the gates are lifted immense scour must take place and any deposit be rapidly removed. The sluiceway is in fact a large silt trap.

139. Weir Sluices of Corbett Dam. The weir sluices of the Corbett dam on the Shoshone River, Wyoming, are given in Figs. 109, 110, and 111.

The canal takes out through a tunnel, the head of which has necessarily to be recessed far behind the location of the weir sluices. Unless special measures were adopted, the space between the sluice gates and the tunnel head would fill up with sand and deposit and block the entrance.

To obviate this a wall 8 feet high is built encircling the entrance. A "divide" wall is also run out upstream of the weir sluices, cutting them off from the weir and its approaches. The space between these two walls forms a sluiceway which draws the current of the river in a low stage past the canal head and further forms a large silt trap which can be scoured out when convenient. Only a thin film of surface water can overflow the long encircling wall, then it runs down a paved warped slope which leads it into the head gates, the heavy silt in suspension being deposited in the sluiceway. This arrangement is admirable.

The fault of the weir sluices as built is the narrowness of the openings which consist of three spans of 5 feet. One span of 12 feet would be much more

effective. In modern Indian practice, weir sluices on large rivers are built with 20 to 40 feet openings.

140. Weir Scouring Sluices on Sand. Weir scouring sluices built on pure sand on as large rivers as are met with in India are very formidable works, provided with long aprons and deep lines of curtain blocks. An example is given in Fig. 112 of the so-termed undersluices of the Khanki weir over the Chenab River in the Punjab. The spans are 20 feet, each closed by 3 draw gates, running in parallel grooves, fitted with antifriction wheels (not rollers), lifted by means of traveling power winches which straddle the openings in which the grooves and gates are located.

The Merala weir sluices of the Upper Chenab canal have 8 spans of 31 feet, piers $5\frac{1}{2}$ feet thick, double draw gates 14 feet high.

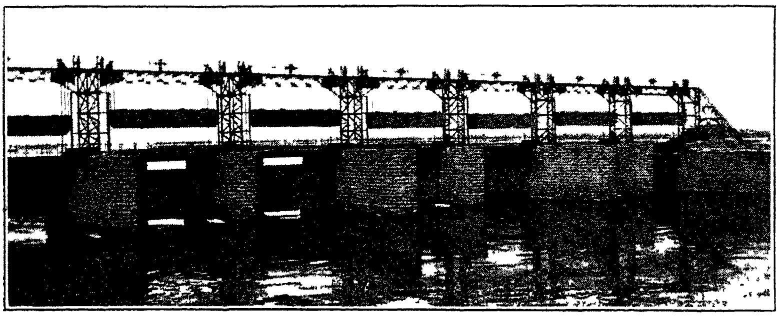


Fig 113 View of Merala Weir Sluices, Upper Chenab Canal

These are lifted clear of the flood, which is 21 feet above floor, by means of steel towers 20 feet high erected on each pier. These carry the lifting apparatus and heavy counterweights. These gates, like those at Laguna weir, Fig. 108, bear against Stoney roller frames.

Fig. 113 is from a photograph of the Merala weir sluices. The work is a partial regulator, in that complete closure at high flood is not attempted. The Upper Chenab canal is the largest in the world with the sole exception of the Ibramiyah canal in Egypt, its discharge being 12,000 second-feet. Its depth is 13 feet. The capacity of the Ibramiyah was 20,000 second-feet prior to head regulation.

141. Heavy Construction a Necessity. In works of this description solid construction is a necessity. Light reinforced concrete construction would not answer, as weight is required, not only

to withstand the hydrostatic pressure but the dynamic effects of flood water in violent motion. Besides which widely distributed

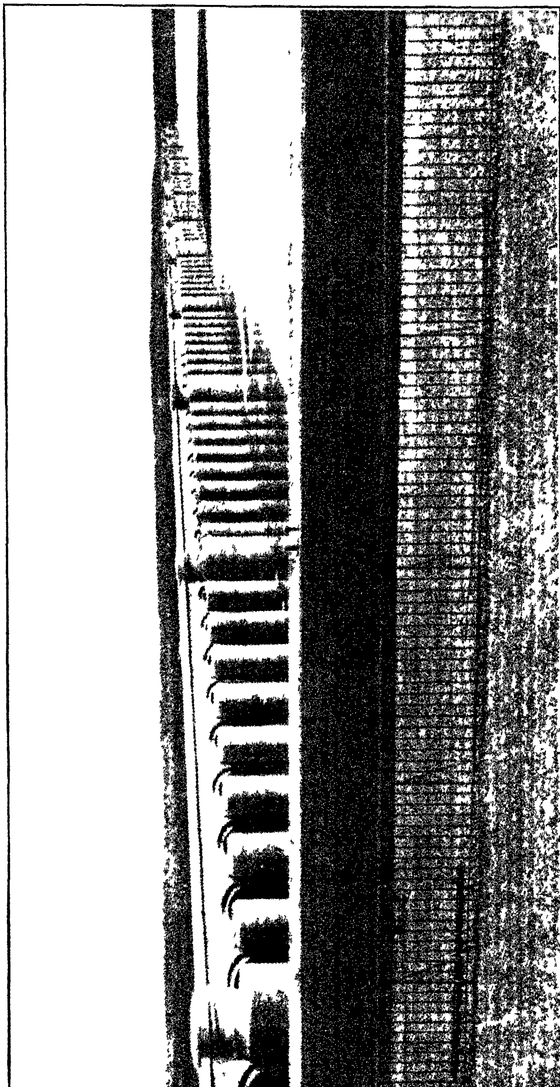


Fig 114 View of the Assiut Barrage across the Nile

weight is undoubtedly necessary for works built on the shifting sand of a river bed, although this is a matter for which no definite rules can be formed.

The weir sluices at Laguna and also at the Corbett dam, are solid concrete structures without reinforcement.

In the East, generally, reinforced concrete is not employed nor is even cement concrete except in wet foundations, the reason being that cement, steel, and wood for forms are very expensive items whereas excellent natural hydraulic lime is generally available, skilled and unskilled labor is also abundant. A skilled mason's wages are about 10 to 16 cents and a laborer's 6 to 8 cents for a 12-hour day. Under such circumstances the employment of reinforced cement concrete is entirely confined to siphons where tension has to be taken care of.

In America, on the other hand, the labor conditions are such that reinforced concrete which requires only unskilled labor and is mostly made up by machinery, is by far the most suitable form of construction from point of view of cost as well as convenience.

This accounts for the very different appearance of irrigation works in the East from those in the West. Both are suitable under the different conditions that severally exist.

142. Large Open Dams across Rivers. Of open dams built across rivers, several specimens on a large scale exist in Egypt. These works, like weir sluices are partial regulators and allow free passage to flood water.

Assiut Barrage. In the Assiut barrage, Figs. 114 and 115,* constructed across the Nile above the Ibramiyah canal head in lower Egypt, the foundations are of sand and silt of a worse quality than is met with in the great Himalayan rivers. The value of c adopted for the Nile is 18, against 15 for Himalayan rivers. This dam holds up 5 meters of water, the head or difference of levels being 3 meters. Having regard to uplift, the head is the difference of levels but when considering overturning moment, on the piers, $\frac{H^3}{6}-\frac{h^3}{6}$ is the moment, H and h being the respective depths of water above and below the gates. It is believed that in the estimation of the length of travel the vertical sheet piling was left out of consideration. Inspection of the section in Fig. 115

*In Figs. 115 and 116 and in the discussion of these problems in the text, the metric dimensions used in the plans of the works have been retained. Meters multiplied by the factor 3.28 will give the proper values in feet.

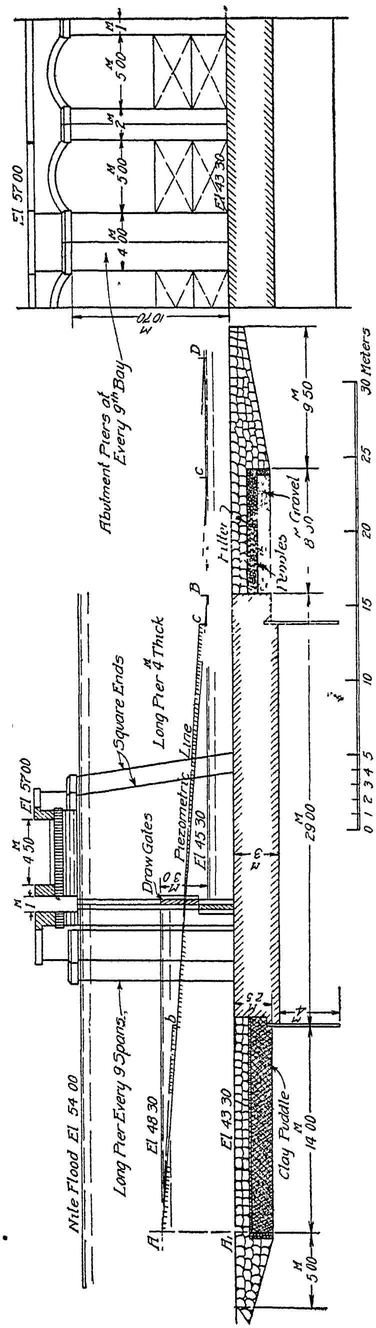


Fig 115 Profile of the Assiut Barrage

shows that the foundation is mass cement concrete, 10 feet deep, on which platform the superstructure is built. This latter consists of 122 spans of 5 meters, or 16 feet, with piers 2 meters thick, every ninth being an abutment pier 4 meters thick and longer than the rest. This is a work of excessive solidity the ratio of thickness of piers to the span being .48, a proportion of .33 S would, it is considered be better. This could be had by increasing the spans to 6 meters, or 20 feet right through, retaining the pier thickness as it is at present.

143. General Features of River Regulators. All these river regulators are built on the same general lines, viz, mass foundations of a great depth, an arched highway bridge, with spring of arch at flood level, then a gap left for insertion of the double grooves and gates, succeeded by a narrow strip of arch sufficient to carry one of the rails of the traveling winch, the other resting on the one parapet of the bridge.

The piers are given a batter downstream in order to better distribute the pressure on the foundation. The resultant of the weight of one span combined with the horizontal water

pressure must fall within the middle third of the base of the pier, the length of which can be manipulated to bring this about. In this case it does so even with increase of the span to 6 meters. This combined work is of value considered from a military point of view, as affording a crossing of the Nile River; consequently the extreme solidity of its construction was probably considered a necessity.

In some regulators girders are substituted for arches, in others as we have noted with regard to the Merala weir sluices, the superstructure above the flood line is open steel work of considerable height.

144. Stability of Assiut Barrage. The hydraulic gradient in Fig. 115, neglecting the vertical sheet piling, is drawn on the profile and is the line AB, the horizontal distance is 43 meters while the head is 3 meters. The slope is therefore 1 in $14\frac{1}{3}$. The uplift is the area enclosed between AB and a horizontal through B which is only 1.4 meters at its deepest part near the gates. Upstream of the gates the uplift is more than balanced by the weight of water overlying the floor. The horizontal travel of the percolation is from A to B plus the length of the filter as explained in section 135. The horizontal travel is therefore $51\frac{1}{2}$ meters and the ratio $\frac{L}{H}$, or c, is $\frac{51.5}{3}=17.2$. The piezometric line has also been shown, including in this case the two vertical obstructions. Their effect on the uplift is very slight, owing to the fore curtain which raises the grade line. The slope in this case is obtained by adding the vertical to the horizontal travel, i.e., from B to D, BC and CD being 8 meters each in length, AD is then the hydraulic gradient which is 1 in 23. Steps occur at points b and c; for instance the line AB is part of AD, the line bc is parallel to AD drawn up from C, and the line cB is similarly drawn up from B forming the end step.

This work is the first to be built with a filter downstream, which has the practical effect of adding to the length of percolation travel irrespective of the hydraulic grade.

145. The Hindia Barrage. The Hindia barrage, quite recently erected over the Euphrates River near Bagdad, is given in Fig. 116. This work, which was designed by Sir William Willocks, bears a

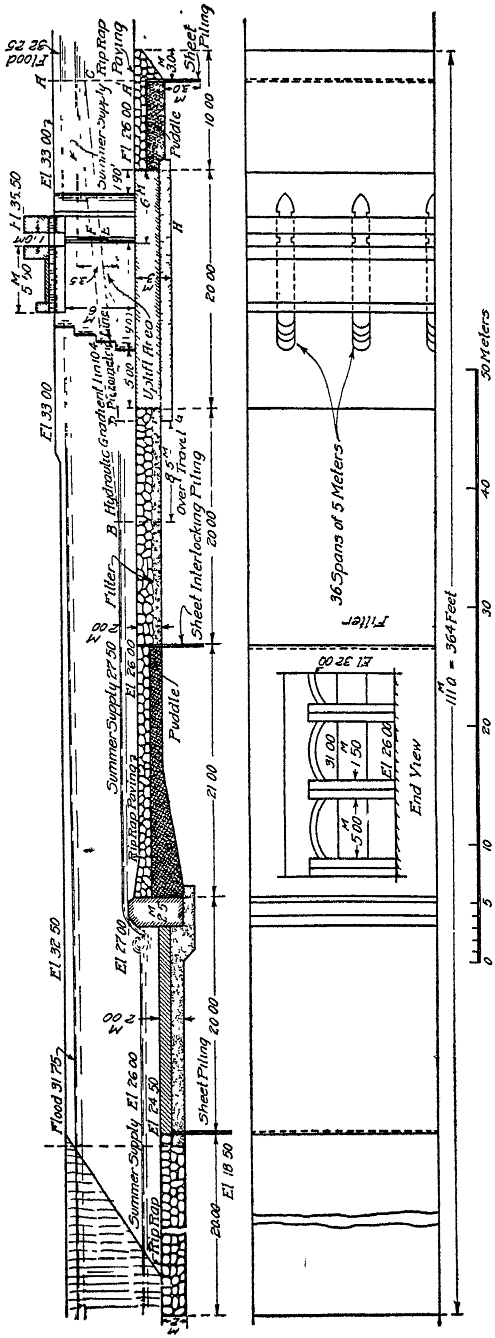


Fig 116 Plan and Section of the Hindia Barrage across the Euphrates River

close resemblance to the Egyptian regulators, viz, the Assiut, the Zifta, and other works constructed across the river Nile. The piers are reduced to 1.50 meters from the 2-meters thickness in the Assiut dam, Fig. 115, and there are no abutment piers, consequently the elevation presents a much lighter appearance. The ratio of thickness to span is .3. In order to reduce the head on the work, a filter bed 20 meters wide is introduced just beyond the platform of the foundation of the regulating bridge. The upward pressure is thus presumed to be nil at the point *D*. The head is the distance between the summer supply level upstream, and that downstream above the subsidiary weir, this amounts to 3.50 meters. The length of compulsory travel

from A to B including .50 meter due to the sheet piling is 36.50 meters. AB is then the hydraulic gradient, which is 1 in $\frac{36.5}{3.5}$ $=1$ in 10.4. The piezometric line DFC is drawn up from D parallel to AB. The area of uplift is $DGHEF$; that part of the uplift below the line DE is however accounted for by assuming all masonry situated below *El.* 27.50 as reduced in weight by flotation, leaving the area DEF as representing the uplift still unaccounted for.

Beyond the filter is a 21-meter length of impervious apron consisting of clay puddle covered by stone paving, which abuts on a masonry subsidiary weir. This wall holds the water up one meter in depth and so reduces the head to that extent, with the further addition of the depth of film passing over the crest at low water which is .5 meter, total reduction 1.50 meters.

This is the first instance of the use of puddle in a fore apron, or talus; its object is, by the introduction of an impervious rear apron 21 meters long, to prevent the subsidiary weir wall from being undermined. The head being $1\frac{1}{2}$ meters, the length of travel required, taking c as 18, will be $18\times1.5=27$ meters. The actual length of travel provided is vertical 15, horizontal 41, total 56 meters, more than double what is strictly requisite. The long hearth of solid masonry which is located below the subsidiary drop wall is for the purpose of withstanding scour caused by the overfall. Beyond this is the talus of riprap 20 meters wide and a row of sheet piling. The total length of the floor of this work is 364 feet, with three rows of sheet piles. That of the Assiut barrage is 216 feet with two rows of sheeting. The difference in head is half a meter only, so that certain unknown conditions of flood or that of the material in the bed must exist to account for the excess.

146. American vs. Indian Treatment. In American regulating works it is generally the fashion where entire closure is required to locate the draw gates and their grooves inside the panel or bulkhead wall that closes the upper part of the regulator above the sluice openings. Thus when the gates are raised they are concealed behind the panel walls. In Indian practice the gate grooves in the piers are generally located outside the bulkhead wall; thus when hoisted, the gates are visible and accessible. Fig. 117 is from a

photograph of a branch head, illustrating this. The work is of reinforced concrete as can be told from the thinness of the piers. In an Indian work of similar character the pier noses would project well beyond the face wall of the regulator and the gates would be raised in front, not behind it.

The use of double gates is universal in Eastern irrigation works; they have the following unquestionable advantages over a single gate: *First,* less power for each is required to lift two gates than one; *second,* when hoisted they can be stacked side by side and so the pier can be reduced in height; *third,* where sand or silt is in suspension, surface water can be tapped by leaving the lower leaf down while the upper is raised; and *fourth,* regulation is made easier.

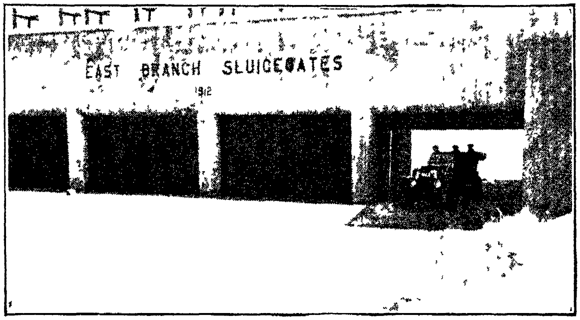


Fig 117 Typical American Regulating Sluices in Reinforced-Concrete Weir

In the Khanki weir sluices, Fig. 112, 3 gates running in 3 grooves are employed.

147. Length of Spans. In designing open dams the spans should be made as large as convenient, the tendency in modern design is to increase the spans to 30 feet or more; the Laguna weir sluices are 33½ feet wide and the Merala 31 feet. The thickness of the piers is a matter of judgment and is best expressed as some function of the span, the depth of water by which the height of the piers is regulated, forms another factor.

The depth of water upheld regulates the thickness more than the length. The length should be so adjusted that the resultant line of pressure combined of the weight of one pier and arch, or superstructure and of the water pressure acting on one span falls within the middle third of the base.

For example take the Assiut regulator, Fig. 115. The contents of one pier and span allowing for uplift is roughly 390 cubic meters of masonry, an equivalent to 1000 tons. The incidence of W is about 2 meters from the middle third downstream boundary.

The moment of the weight about this point is therefore $1000 \times 2 = 2000$ meter tons. Let H be depth of water upstream, and h downstream, then the overturning moment is expressed by $\frac{(H^3 - h^3)wl}{6}$. Here $H=5$, $h=2$ meters, $w=1.1$ tons per cubic meter, the length l of one span is 7 meters; then the moment $= \frac{(125-8) \times 1.1 \times 7}{6} = 150$ meter tons. The moment of resistance is therefore immensely in excess of the moment of water pressure. The height of the pier is however governed by the high flood level, the width by the necessity of a highway bridge. At full flood nearly the whole of the pier will be immersed in water and so lose weight. There is probably some intermediate stage when the water pressure will be greater than that estimated, as would be the case if the gates were left closed while the water topped them by several feet, the water downstream not having had time to rise to correspond.

148. Moments for Hindia Barrage. In the case of the Hindia barrage, Fig. 116, $H=5$ meters, $h=1.5$, then

$$M = \frac{(125-3\ 4) \times 1.1 \times 6.50}{6} = 145 \text{ meter tons}$$

The weight of one span is estimated at 180 tons. Its moment about the toe of the base is about $180 \times 6.5 = 1170$ meter tons. The factor of safety against overturning is therefore $\frac{1170}{145} = 8$. The long base of these piers is required for the purpose of distributing the load over as wide an area as possible in order to reduce the unit pressure to about one long ton per square foot. This is also partly the object of the deep mass foundation. The same result could doubtless be attained with much less material by adopting a thin floor say two or three feet thick, reinforced by steel rods so as to ensure the distribution of the weight of the superstructure evenly over the whole base. It seems to the writer that the Assiut barrage with its mass foundation having been a success

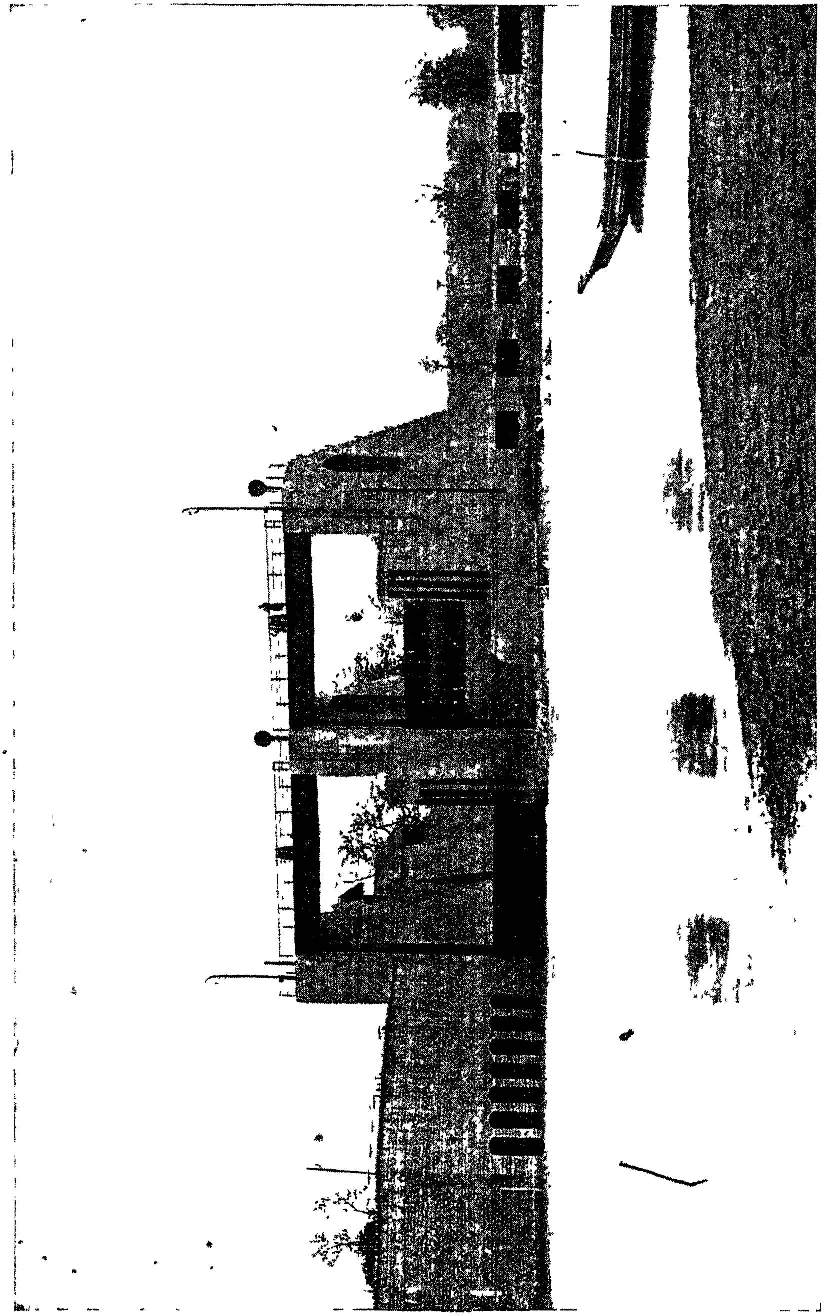


Fig. 118. Head Regulator and Undersluices of North Mon Canal in Burma, Showing Portion of Weir

TABLE II

Pier Thickness—Suitable for Open Partial Regulators and Weir Sluices

SPAN	DEPTHS OF WATER							
	15 FEET		20 FEET		25 FEET		30 FEET	
	M	*T*	*M*	*T*	*M*	*T*	*M*	*T*
10 feet	25	2 5	27	2 7	29	2 9	31	3 1
15 feet	24	3 6	.26	3 9	.28	4 2	30	4 5
20 feet	23	4 6	25	5 0	27	5 4	.29	5 8
25 feet	21	5 3	24	6 0	26	6 5	28	7 0

M is multiplier of span for thickness *T*

as regards stability, is no reason why a heavy style of construction such as this should be perpetuated.

149. North Mon Canal. In Fig. 118 is shown the head works of the Mon right canal in Burma. The weir is of type A, with crest shutters and sluices of large span controlled by draw gates. In the canal head, the gates are recessed behind the face wall as in American practice.

150. Thickness of Piers. Table II, though purely empirical will form a useful guide of thickness of piers in open dams or partial regulators.

If reinforced, very considerable reduction can be made in the thickness of piers, say $\frac{2}{3}$, but for this class of river work a heavy structure is obligatory.

151. Advantages of Open Dams. Open dams have the following advantages over solid weirs, or combinations of solid and overfall dams: First, the river bed is not interfered with and consequently the heading up and scour is only that due to the obstruction of the piers, which is inconsiderable. This points to the value of wide spans. Second, the "river low" supply is under complete control. Third, a highway bridge across the river always forms part of the structure which in most countries is a valuable asset.

Open dams, on the other hand, are not suitable for torrential rivers as the Himalayan rivers near their points of debouchure from the mountains, or wherever such detritus as trees, logs, etc., are carried down in flood time.

152. Upper Coleroon Regulator. Fig. 119 is from a photograph of a regulating bridge on the upper Coleroon River in the Madras Presidency, southern India. Originally a weir of type A was constructed at this site in conjunction with a bridge. The constriction of the discharge due to the drop wall, which was six feet high, and the piers of the bridge, caused a very high afflux and great scour on the talus. Eventually the drop wall was cleared away altogether, the bridge piers were lengthened upstream and fitted with grooves and steel towers, and counterweighted draw gates some 7 feet deep

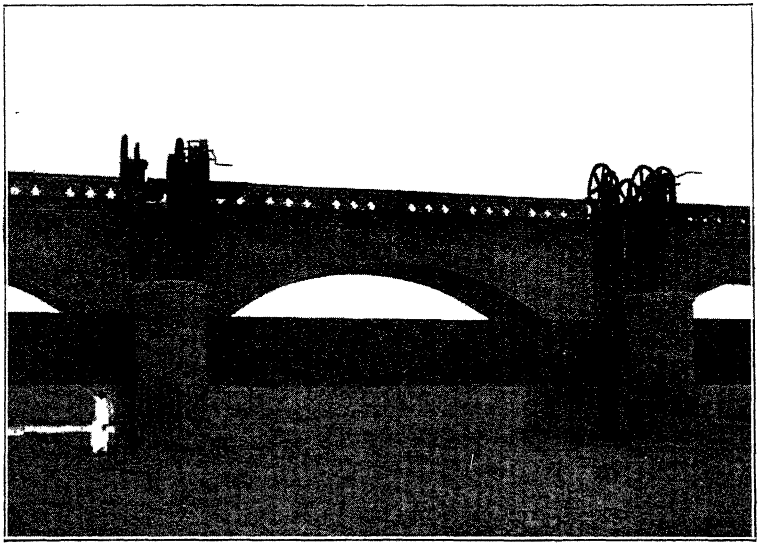


Fig. 119. View of Regulating Bridge on the Upper Coleroon River, Southern India

took the place of the drop wall. In the flood season the gates can be raised up to the level of the bridge parapet quite clear of the flood. The work was thus changed from one of a weir of type A, to an open dam. The original weir and bridge were constructed about half a century ago.

153. St. Andrew's Rapids Dam. Another class of semi-open dam consists of a permanent low floor or dwarf weir built across the river bed which is generally of rock, and the temporary damming up of the water is effected by movable hinged standards being lowered from the deck of an overbridge, which standards support

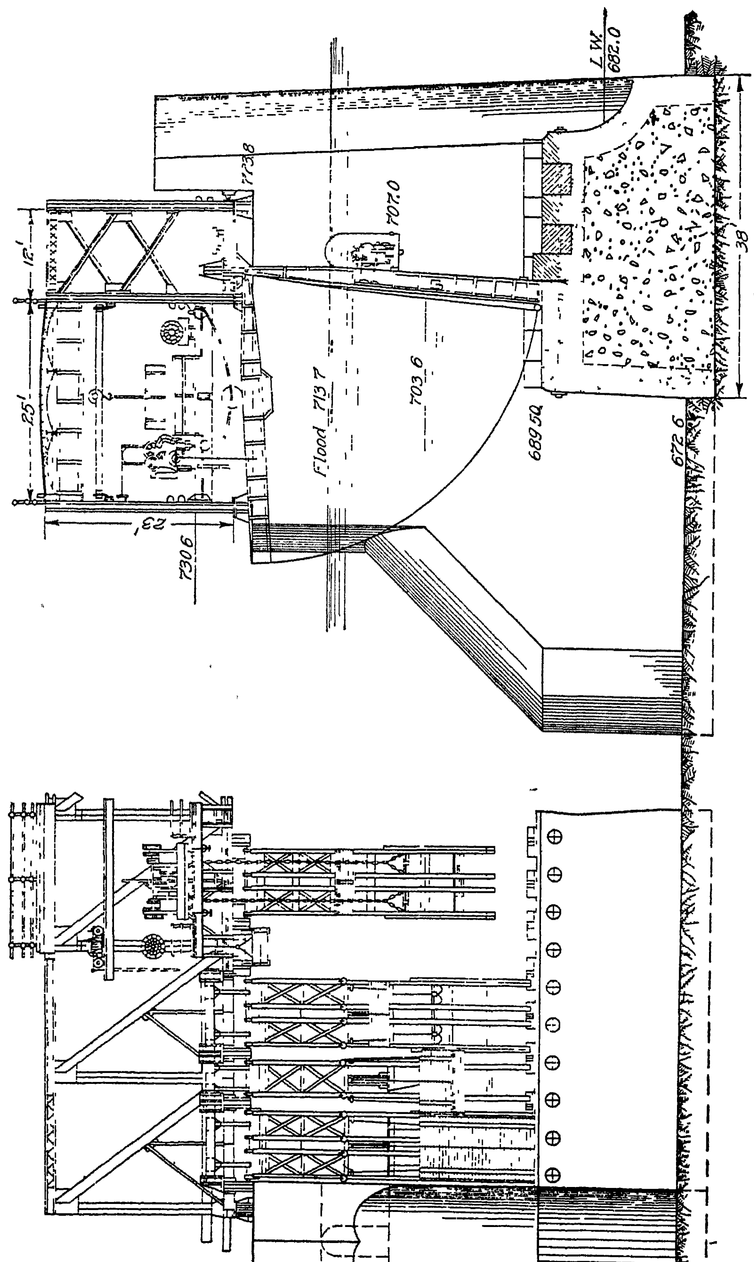


Fig 120 Elevations and Part Section of St. Andrew's Rapids Semi-Open Dam

either a rolled reticulated curtain let down to cover them or else a steel sliding shutter mounted on rollers.

The St. Andrew's Rapids dam, Fig. 120, a quite recent construction, may be cited as an example. The object of the dam is to raise the water in the Red River, Manitoba, to enable steamboats to navigate the river from Winnipeg City to the lake of that name. To effect this the water level at the rapids has to be raised 20 feet above L. W. L. and at the same time, on account of the accumulation of ice brought down by the river, a clear passage is a necessity. The Red River rises in the South, in the State of North Dakota where the thaw sets in much earlier than at Lake Winnipeg, consequently freshets bring down masses of ice when the river and lake are both frozen.

Caméré Type of Dam. The dam is of the type known as the Caméré curtain dam, the closure being effected by a reticulated wooden curtain, which is rolled up and down the vertical frames thereby opening or closing the vents. It is a French invention, having been first constructed on the Seine. The principle of this movable dam consists in a large span girder bridge, from which vertical hinged supports carrying the curtain frames are let drop on to a low weir. When not required for use these vertical girders are hauled up into a horizontal position below the girder bridge and fastened there. In fact, the principle is very much like that of a needle dam. The river is 800 feet wide, and the bridge is of six spans of 138 feet.

The bridge is composed of three trusses, two of which are free from internal cross-bracing, and carry tram lines with all the working apparatus of several sets of winches and hoists for manipulating the vertical girders and the curtain; the third truss is mainly to strengthen the bridge laterally, and to carry the hinged ends of the vertical girders.

It will be understood that the surface exposed to wind pressure is exceptionally great, so that the cross-bracing is absolutely essential, as is also the lateral support afforded by a heavy projection of the pier itself above floor level.

In the cross-section it will be seen that there is a footbridge opening in the pier. This footbridge will carry winches for winding and unwinding the curtains, and is formed by projections thrown

out at the rear of each group of frames. It will afford through communication by a tramway. The curtains can be detached altogether from the frames and housed in a chamber in the pier clear of the floodline.

The lower part of the work consists of a submerged weir of solid construction which runs right across the river; its crest is 7 feet 6 inches above L. W. L. at *El.* 689.50. The top of the curtains to which water is upheld is *El.* 703.6, or 14 feet higher. The dam actually holds up 31 feet of water above the bed of the river.

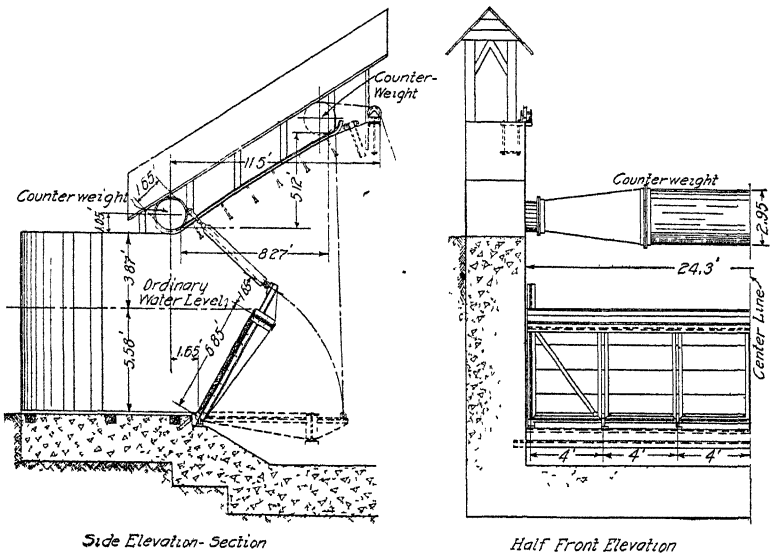


Fig 121 Lauchli Automatic Sluice Gate

This system is open to the following objections: First, the immense expense involved in a triple row of steel girders of large span carrying the curtains and their apparatus; and second, the large surface exposure to wind which must always be a menace to the safety of the curtains.

It is believed that the raising of the water level could be effected for a quarter of the cost if not much less, by adopting a combination of the system used in the Folsam weir, Fig. 50, with that in the Dhukwa weir, Fig. 52, viz, hinged collapsible gates which could be pushed up or lowered by hydraulic jacks as required. The existing lower part of the dam could be utilized and a subway constructed

through it for cross communication and accommodation for the pressure pipes, as is the case in the Dhukwa weir. This arrangement which is quite feasible would, it is deemed, be an improvement on the expensive, complicated, and slow, Caméré curtain system.

154. Automatic Dam or Regulator. Mr. Lauchli of New York, writing for Engineering News, describes a new design for automatic regulators, as follows:

In Europe there has been in operation for some time a type of automatic dam or sluice gate which on account of its simplicity of construction, adapt-

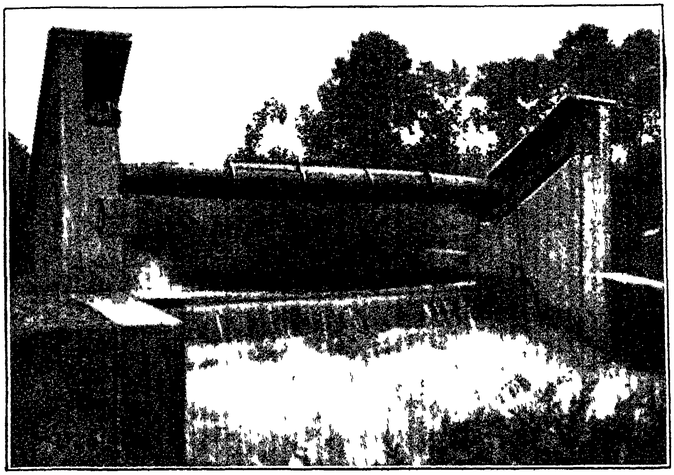


Fig 122. View of Lauchli Automatic Dam Which Has Been for Several Years in Successful Operation in Europe

ability to existing structures, exact mathematical treatment, and especially its successful operation, deserves to attract the attention of the hydraulic engineer connected with the design of hydroelectric plants or irrigation works Fig. 121 shows a cross-section and front elevation of one of the above-mentioned dams now in course of construction, and the view in Fig 122 gives an idea of a small automatic dam of the same type which has been in successful operation for several seasons, including a severe winter, and during high spring floods.

Briefly stated, the automatic dam is composed of a movable part or panel, resting at the bottom on a knife edge, and fastened at the top to a compensating roller made of steel plate and filled with concrete. This roller moves along a track located at each of its ends, and is so designed as to take, at any height of water upstream, a position such as will give the apron the inclination necessary for discharging a known amount of water, and in so doing will keep the upper pool at a constant fixed elevation.

With the roller at its highest position the panel lies horizontally, and the full section is then available for discharging water. Any débris, such as

trees, or ice cakes, etc., will pass over the dam without any difficulty, even during excessive floods, as the compensating roller is located high above extreme flood level.

The dam now in course of construction is located on the river Grafenauer Ohe, in Bavaria, and will regulate the water level at the intake of a paper mill, located at some distance from the power house. The dam has a panel 24 27 ft. long, 6.85 ft. high, and during normal water level will discharge 1400 cu. ft. per sec., while at flood time it will pass 3,530 cu. ft. per sec. of water. As shown in Fig. 121, the main body of the dam is made of a wooden plank construction laid on a steel frame. The panel is connected with the compensating roller at each end by a flexible steel cable wound around the roller end, and then fastened at the upper part of the roller track to an eyebolt. A simple form of roof construction protects the roller track from rain and snow. The panel is made watertight at each extremity by means of galvanized sheet iron held tight against the abutments by water pressure. This type of construction has so far proved to be very effective as to watertightness.

It may be needless to point out that this type of dam can also be fitted to the crest of overflow dam of ordinary cross-section, and then fulfill the duty of movable flashboards.

The probability is that this type will become largely used in the future. A suggested improvement would be to abolish the cross roller having instead separate rollers on each pier or abutment, working independently. There will then be no practical limit to the span adopted.

TYPICAL IRRIGATING WEIR AND LATERALS FOR ORANGE GROVE AT REDLANDS, CALIFORNIA

Photo by Underwood and Underwood, New York City

IRRIGATION ENGINEERING

INTRODUCTION

Problem of Irrigation. The control and distribution of water for irrigation presents to the engineer problems somewhat similar to those of the control and distribution of water for domestic and manufacturing purposes in large cities and towns. The water must be diverted from a flowing stream at a sufficient elevation to command the territory to be irrigated, or it must be impounded in reservoirs at a season of floods or of unusual flow due either to the more-or-less regular recurrence of rainy seasons, or to the melting of snow and ice. Again, it may be derived from subterranean sources, either deep or shallow wells, lifted to the proper elevation by pumps, and applied directly to the land or stored in reservoirs for future use.

The principal difference between securing a water supply for domestic and for irrigation purposes is that, in the former case, the water must be as pure as possible, while in the latter, the impurities that gather in ponds and streams may not be detrimental for irrigation purposes. The sewage of many cities, which has a distinct value as a fertilizer, for example, is used successfully for purposes of irrigation.

Except where water is scarce and difficult to produce at a reasonable cost, and on account of the large quantities of water required for irrigation, it is not necessary to make such expensive provision per unit of volume for the distribution of water for irrigation purposes as is the case for domestic supply. In the majority of cases water for irrigation flows in open channels. However, to guard against loss by evaporation and seepage, it may be necessary to distribute the water through a system of underground or enclosed pipes or conduits.

Crude Methods No Longer Adopted. Irrigation works in the West range from crude and simple ditches, taking their supplies from mountain brooks where the water has been diverted by means of small brush dams, to great masonry walls, blocking the outlet of

TABLE I

Units of Measure

1 Second-Foot	=448 8 gallons per minute
1 Cubic Foot	=7 48 gallons
1 Cubic Foot	=62 4 pounds, at average temperature
1 Second-Foot (running for 12 hours)	=43,200 cubic feet (about one acre-foot)
1,000,000 Cubic Feet	=23 acre-feet (approximately)
1,000,000 Gallons	=3 07 acre-feet
100 California Inches (running for 24 hours)	=3.967 acre-feet
100 Colorado Inches (running for 24 hours)	=$5\frac{1}{6}$ acre-feet
50 California Inches	=1 second-foot (statute)
38 4 Colorado Inches	=1 second-foot (statute)
1 Colorado Inch	=17,000 gallons, in 24 hours (approximately)
1 Second-Foot	=59 5 acre-feet, in 30 days
2 Acre-Feet	=1 second-foot per day (approximately)
1 Acre-Foot	=25 2 California inches, in 24 hours

deep canyons and holding back the water which is to be transported through canals, pipes, or flumes to lands situated many miles away. On account of the increasing scarcity of the water available for irrigation the more crude forms of getting the water to the land are not being installed any more. Practically all water that can be diverted to the land by the cruder methods has been appropriated.

Units of Measure. In making a study of irrigation problems, it is necessary to express the flow or quantity of water in certain units. Those most commonly employed and the relations existing between them are given in Table I. Certain approximate relations that are easily remembered are also indicated.

FACTORS OF WATER SUPPLY

SOURCES OF WATER

PRECIPITATION

Rain, Snow, Hail, and Dew. The supply of water has its source primarily in the precipitation that falls to the earth in the form of rain, snow, hail, or dew. The last two forms of precipitation furnish but a small amount of the supply, while the first two furnish by far the greater part. Precipitation in the form of snow in the mountains furnishes a large part of the irrigation water which is

used directly from the flowing streams, while it also furnishes a good portion of the water stored for irrigation purposes in the early spring run-off; and precipitation in the form of rain furnishes the rest of the storage water.

The moisture of the atmosphere has been evaporated from water and land areas, and from surfaces of plant foliage and other objects. The water, after it reaches the earth, may remain for a time in the form of ice and snow, may run off of the surface directly, or may soak into the soil and furnish the supply for springs, wells, and galleries.

Rainfall Influences. In any region where the climate and soil are adapted to the production of crops, the necessity for irrigation will depend upon the amount of precipitation available, the season of the year when this precipitation is available, and the manner in which the precipitation reaches the earth. The available precipitation cannot be judged, however, from the total annual amount. Where the annual precipitation is less than 20 inches, irrigation is assumed to be necessary. The arid region of this country is usually considered as including that area in which the annual precipitation is below 20 inches—or most of the territory west of the 97th meridian of longitude. The latitude has considerable influence also. Other things being equal, it will require a greater amount of precipitation to produce a crop in the South than in the North. The crops mature more quickly and the evaporation is less in the North.

As illustrating seasonal influence, irrigation is necessary in Italy, because, while the annual precipitation averages about 40 inches, most of this occurs during the winter months or at times other than the agricultural or cropping season. In certain parts of India, the rainfall is as high as 100 to 300 inches per annum; and yet nearly all of this occurs in one or two seasons of the year, and the actual rainfall during the winter months, when most of the cropping is done, may be as low as 5 or 10 inches. The cropping season in the arid West may be taken as occurring between April and August, inclusive, and this constitutes the driest season of the year for a large portion of the West, while it is the wettest for the Northwest.

In referring to the lands of the United States, those of the extreme West are usually called arid; those between the Mississippi Valley and the Rocky Mountains, where the rainfall is occasionally

TABLE II

Precipitation by River Basins in the Arid Region of the United States

Station	Altitude (ft.)	Precipitation (in.)	
		Mean Annual	April–August Inclusive
Rio Grande River—			
Summit, Colorado	11,300	30.75	14.11
Fort Lewis, Colorado	8,500	18.33	7.32
Fort Garland, Colorado	7,937	13.68	7.66
Saguachi, Colorado	7,740	7.97	5.19
Santa Fe, New Mexico	7,026	14.49	8.08
Fort Wingate, New Mexico	6,822	15.38	6.81
Las Vegas, New Mexico	6,418	18.56	12.01
Albuquerque, New Mexico	5,032	7.52	4.13
Socorro, New Mexico	4,560	8.44	4.90
Deming, New Mexico	4,315	9.97	5.20
Gila River—			
Fort Bayard, New Mexico	6,022	15.25	7.84
Prescott, Arizona	5,389	17.20	7.66
Fort Apache, Arizona	5,050	17.61	7.80
Fort Grant, Arizona	4,914	14.93	7.46
Phoenix, Arizona	1,068	7.87	2.61
Texas Hill, Arizona	353	3.47	.66
Yuma, Arizona	141	3.10	.60
Platte River—			
Pike's Peak, Colorado	14,134	29.55	16.92
Fort Saunders, Wyoming	7,180	12.10	7.48
Fort Fred Steele, Wyoming	6,850	9.05	4.31
Cheyenne, Wyoming	6,105	13.60	9.31
Colorado Springs, Colorado	6,010	14.44	10.72
Denver, Colorado	5,241	14.02	9.14
Fort Morgan, Colorado	4,500	12.96	9.33
Missouri River—			
Virginia, Montana	5,480	15.21	8.92
Fort Ellis, Montana	4,754	21.07	18.80
Helena, Montana	4,266	12.77	6.96
Fort Shaw, Montana	2,550	10.88	6.97
Poplar, Montana	1,955	13.51	8.66

sufficient to mature the crops, are designated as semi-arid or semi-humid; and the lands to the east of the Mississippi Valley, over which the rainfall is generally sufficient to mature the crops, are spoken of as humid. This distinction is based largely upon the amount of precipitation during the crop-growing season. On this basis, the humid portion of the United States embraces those regions over which the precipitation during the cropping season is from 10 to 15 inches, depending upon the character of the soil and other

TABLE III

Mean Annual Precipitation in the United States

Meteorological Districts	Annual Precipitation (in.)
New England	41.33
Middle Atlantic States	41.64
South Atlantic States	50.99
Florida Peninsula	50.68
East Gulf States	52.68
West Gulf States	38.15
Ohio Valley and Tennessee	42.96
Lower Lake Region	34.86
Upper Lake Region	31.77
North Dakota	19.45
Upper Mississippi Valley	33.62
Missouri Valley	29.27
Northern Slope	15.60
Middle Slope	22.76
Southern Slope	22.77
Southern Plateau	8.82
Middle Plateau	11.59
Northern Plateau	14.81
North Pacific Coast Region	48.33
Middle Pacific Coast Region	26.75
South Pacific Coast Region	14.00

modifying conditions. Table II shows in a general way the amount and extent of the precipitation over portions of the arid region.

Comparison by Meteorological Districts. The Weather Bureau has divided the United States into twenty-one districts for meteorological purposes, and the precipitation averages for these districts are given in Table III. In each meteorological district the general law and average amount of precipitation are practically uniform; but the variation of precipitation at different stations in some districts is as pronounced as the variation between the extreme district means. The factor most influential in determining the amount of rainfall in a given district is the proximity or other relation to mountain ranges and to the sea or other large body of water. Thus the warm, moist winds of the North Pacific lose a large portion of their moisture upon the western slopes of the Sierra Nevada and the Cascade ranges, so that little is left for the plateau to the east of these ranges. The winds blowing over the Gulf Stream into the South Atlantic and Gulf States yield their moisture to them, so that, as they ascend the valley of the Mississippi and its tributaries, their

moisture and consequent precipitation decrease. The departure of the Gulf Stream from the coast north of Cape Hatteras, causes its influence to be felt to a less extent in precipitation in the North Atlantic States.

It will be noted that the average rainfall over the northern portion of the Pacific Coast would be sufficient for the production of crops, provided it fell during the proper season of the year. Other areas may be noted over which the annual rainfall is apparently sufficient for the maturing of crops. That the amount of precipitation is greatly influenced by altitude, is shown by comparing the relative amounts of precipitation at places having the same latitude. Thus in the region between Reno, Nevada, and San Francisco, California, the average annual precipitation in the Sacramento Valley is about 15 inches; while to the eastward the precipitation increases in amount with the altitude, until, along the summits of the mountains, it averages from 50 to 60 inches. Still farther east, the precipitation again diminishes with the decreasing altitude, until, in Nevada, it varies from 5 to 10 inches. All through the West, precipitation in the high mountains is much in excess of that in the adjacent low valley lands. As a result, while the precipitation is often insufficient to mature the crops in the lowlands, sufficient precipitation occurs in the mountains to furnish a constant supply for the perennial discharge of streams or for the filling of storage reservoirs.

Importance of Consideration. Precipitation, being the primary source of all water supplies, is the basis of calculation of the amount of water available from whatever source; and consideration of the amount and intensity of rainfall is a necessary preliminary to the design and construction of storage works, whether for purposes of irrigation or for domestic supply. The amount of rain that will fall at any one place, in any day, month, or year, cannot be predicted with certainty by any method known to science. A record of past rainfalls, however, will afford a guide to our judgment in estimating the probable amount of future rainfalls, and, in fact, forms practically the only basis for such judgment. The total annual rainfall is seldom the same for any two years at the same place, or at any two places for the same year, and these variations seem to follow no definite law. A season which is dry at one place, may have con-

ditions entirely the reverse only a few miles away, and the total annual rainfall of two places only a few miles apart often varies considerably, especially in a mountainous and arid country.

One of the most important considerations in designing irrigation works, and especially storage reservoirs, is the maximum amount of rainfall which may occur. Great floods are the immediate result either of heavy, protracted rainstorms, or of the sudden melting of snow accompanied by rain in the mountains. In nearly all river valleys, there are periods of maximum rainfall, the recurrence and effect of which should be given careful study. The following are examples of unusual precipitation:

In the neighborhood of Yuma, Arizona, the average annual rainfall is about 3 inches, but in the last week of February, 1891, $2\frac{1}{2}$ inches fell in 24 hours

In the neighborhood of San Diego, California, the average annual rainfall is about 12 inches; but in the storms of 1891, 13 inches fell in 23 hours, and $23\frac{1}{2}$ inches in 54 hours

The average annual discharge of the Salt River in Arizona is about 1000 second-feet, and the average flood discharge is about 10,000 second-feet, yet, as the result of an unusually violent rainstorm in the spring of 1890, the flood discharge amounted to 140,000 second-feet. A year later, as the result of a still more violent storm, the discharge increased to the enormous amount of nearly 300,000 second-feet.

It is of course out of the question to design irrigation works so that they shall control and safely pass away the flood discharge of unusual storms such as described above. Such cloudbursts may occur once in a lifetime, but the increased expenditure necessary to provide for them is generally unwarranted.

Measurement of Snowfall. Measurements of snow are ordinarily recorded in inches of fall as found upon a level surface free from drifts. It is difficult to obtain an average depth in windy weather, and the best judgment is essential in ascertaining this. Generally, besides expressing the depth in inches, a cylinder of snow of this depth is collected and melted in a tube or can of the same diameter as the cylinder of snow, and the depth of water resulting is recorded as precipitation or rainfall.

Measurement of Rainfall. *Description of Rain Gage.* Measurements of rainfall are usually made in this country by means of the pluviometer, Fig. 1, consisting essentially of a circular cup of thick brass, its rim brought to a chisel edge, the bottom being cone-shaped and connected with a deep tube of known diameter into

which the rain flows from the cup. The area of the top of the cup and that of the tube bear a known relation to each other—usually 10:1—and the depth in the tube is measured by a stick so graduated that when it is lowered to the bottom of the tube, the scale will give the actual depth of rainfall, allowance being made in the scale for both the relative areas and the displacement caused by the stick. The depth is usually expressed in inches and decimals of an inch. The readings should be taken daily and at the beginning and end of each storm.

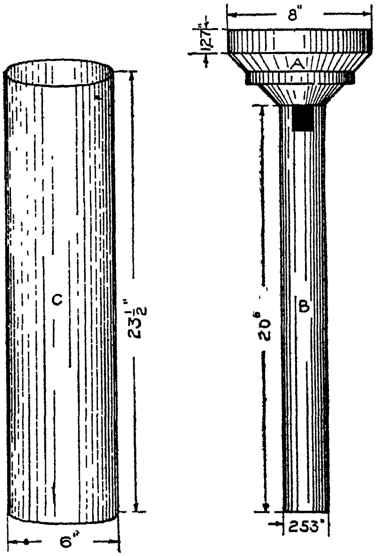


Fig 1 Rain Gage. *A*—Collector, *B*—Receiver, *C*—Overflow Attachment

Conditions Affecting Catchment. The size of the collector cup seems to have some effect upon the catchment. Of four 3-inch cups and one 8-inch cup in use on Mt. Washington, the average total amount collected by the 3-inch cups in one year was 46.26 inches, while that collected by the 8-inch cup was 58.70 inches. The larger the collector cup, probably the more accurate will be the result. The position of the gage relative to the ground surface will also have an important influence upon the amount of catchment, those placed near the surface generally giving the higher results. It has been found that a gage 100 feet above the ground will give on the average only 65 per cent as much rainfall as one upon the surface. The intensity of the wind seems to be the controlling factor in these variations. It is maintained by many that gages at the surface give less accurate results, since they receive not only the actual precipitation, but also a certain amount of moisture from the surrounding ground, which, after falling, again rises by splashing and evaporation, and is once more precipitated. This, however, gives the actual precipitation at the surface of the soil. A large number of gages of the signal service are placed upon the roofs of tall buildings, and in cities this is generally necessary. In open country, a

height of from 3 to 6 feet from the surface will probably give the most accurate results. The gage should be at least as far from any building or other obstacle as the top of this is above the gage, and the rim of the collector cup should be level. The United States Weather Bureau has for years been taking records of precipitation in various parts of the country, and many stations are now so operated, records being received from hundreds of voluntary observers as well.

Self-Recording Gages. For many purposes it is desirable to know the rate of fall for short intervals of 5 minutes or less, and for ascertaining this, self-recording gages are necessary. Several styles of such gages have been used, one of which, the *tipping tank*, tips and empties itself as soon as it has received 0 01 inch of rainfall, immediately returning to an upright position, the time of each discharge being recorded automatically. Another style of gage consists of a tank suspended by a spring balance, a pencil attached to the tank continuously recording its vertical position upon a cylinder revolved by clockwork once in 24 hours. In using any recording gage, the total water caught should be retained, and measured or weighed each day as a check upon the record. The records of the United States Weather Bureau are available to anyone, and should be freely consulted in the study of the precipitation of any locality.

RUN-OFF

By the *yield* or *run-off* of a catchment area, is meant the total amount of water flowing from a given drainage area, generally as streams fed by the rainfall upon such area. This is never the whole of such rainfall.

Surface, River, and Underground Supplies. In the temperate and frigid zones, rain (including snow) is considered the source of water. Whether for domestic or manufacturing purposes or for purposes of irrigation, rain, since it does not fall continuously, must be caught and stored up to tide over periods of longer or shorter duration. In a natural way this is accomplished to a certain extent through the agency of porous soil and rocks, underground caverns, lakes and ponds, glaciers, etc. Artificial storage, on the other hand, is accomplished by means of cisterns and reservoirs. It is the water flowing from some drainage area, as a watershed of thou-

sands of acres, which gives the greater part of the supply for artificial storage. If this drainage area is the surface of the ground, the run-off or yield is called *surface water*.

In falling, some rain is intercepted by the foliage and stems of trees and smaller plants, to be returned later to the air through evaporation. Of that which reaches the earth, a portion flows over the surface, and the remainder enters the soil or evaporates. If the soil is very porous, almost all the rain falling upon it may be absorbed; if non-porous, very little may enter it. All soils, even the densest rocks, are more or less porous, and will absorb water to some extent.

After a rain has ceased, the small streams carry less and less water; but those of any size seldom become entirely dry, though weeks and even months may elapse between rainfalls and the surface of the ground may become very dry. During this time the immediate supply is not the rain, but is that portion of previous rainfalls absorbed by the earth and now being yielded slowly. In general, the more porous the soil, the more water it will receive for this purpose during a given rainfall; and the finer its grain, the more slowly will it yield its supply and become exhausted. The underground flow need not necessarily reach the same stream as the surface flow but the dip of the strata may carry it into another valley. The underground flow frequently emerges as springs; but the larger portion of it generally reaches the stream through the banks, and in some cases through the bottom of the channel.

A study of the material and dip of the strata, and of the surface conditions, such as topographical features, vegetation, location of ponds, etc., as well as of the rainfall and other meteorological conditions, is necessary in forming any estimate of the probable amount and yield of a given watershed, where this cannot be measured directly. The total amount of rainfall reaching the ground is not yielded again by the combined surface and underground flow; a large part of it is lost by evaporation from the surface of the ground, and from the surface of ponds or other bodies of water; much is taken up by vegetation, to be returned to the air by evaporation from the foliage. A portion is held in the soil by capillary attraction. Probably none of the precipitation settles into the lowest strata which have no outlet, since these were filled ages ago. If, however, water

is drawn from these deep strata by wells, the amount thus withdrawn must be replenished, or the supply will ultimately be exhausted.

The dividing line between surface supplies and river supplies is indefinite; but when the supply is taken directly from a river or lake without impounding or storage, it should be called a *river supply* or *lake supply*. The conditions are in many respects similar to those affecting surface waters; but the supply is somewhat more constant and of greater volume, owing to the larger drainage area. Lakes act as regulators of flow, and take the place of artificial storage reservoirs. They are generally but enlargements of a river channel, although some lakes are formed directly from surface flow or from large springs and from the sources of rivers, while still others have ground water as both source and outlet. Lakes can, in most cases, be relied upon as being more constant than rivers in regard to the quantity of water available.

The ground water, or that flowing subterraneously and constituting the *underground supply*, may emerge as springs or be intercepted by wells dug or bored to the porous stratum through which it flows. Such water generally fills the porous stratum throughout, and moves slowly through the interstices down hill. If, in traveling through such a stratum, the ground water encounters a fault, or impervious stratum, this may be followed to the surface, where the water will emerge in the form of springs.

Amount of Run-Off. The run-off of a given catchment area may be expressed as the number of second-feet of water flowing in the stream draining that area, or as the number of inches in depth of a sheet of water spread over the entire catchment, or it may be expressed volumetrically as so many cubic feet or acre-feet, or in percentage of the precipitation.

The amount of run-off depends upon the amount of precipitation, the manner in which it falls, and many other varying climatic and topographic factors. Many formulas, none of which gives uniformly satisfactory results, have been worked out, expressing the relation between precipitation and run-off. The climatic influences bearing most directly on run-off are the total amount of precipitation, its rate of fall, and the temperature of earth and air. When most of the precipitation occurs in a few violent showers, the

percentage of run-off is higher than when the water is given abundant time to enter the soil. If the temperature is high and the wind blowing briskly, much greater loss will occur from evaporation than if the ground is frozen and the air is quiet. Some other factors affecting the run-off are size, shape, and topography of the drainage area, nature of the soil and surface, and condition and growth of the vegetation.

Rate of Run-Off. Within a given drainage area, the rates of run-off vary on the different portions. In a large basin, the rate of run-off for the entire area may be low if the greater portion of the area is nearly level; but at the headwaters of streams, where the

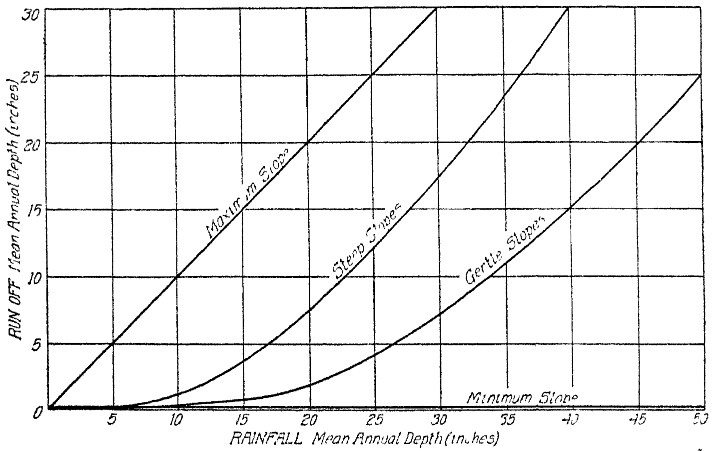


Fig 2 Curve Showing Relation of Run-Off to Rainfall

slopes are steep and perhaps rocky, the rate of run-off will be higher. Other things being equal, the percentage of run-off will increase with the rainfall; and in humid regions, where the rainfall is greatest, the rate of run-off will naturally be highest.

The accompanying diagram, Fig. 2, prepared by Mr. F. H. Newell, illustrates the relation between mean annual run-off and mean annual rainfall. Along the vertical axis is plotted the mean annual run-off in inches for a given drainage area, while the annual rainfall in inches is plotted along the horizontal axis. The diagonal line represents the extreme limit of run-off that would occur upon a steep, smooth, impenetrable surface; the horizontal line at the bottom represents the limit upon a level, porous surface from which

there would be no run-off. The upper curved line represents an average condition in mountainous regions, from which the run-off is large; the lower curved line represents the condition in a catchment area consisting of broad valleys and gentle slopes, from which the run-off is relatively small. For instance, in an area of the latter kind having an annual rainfall of 40 inches, the annual run-off, as indicated by the diagram, will be 15 inches. However, the relation between these two quantities will be largely influenced by the conditions affecting the run-off as stated above.

Volume of Discharge. The maximum discharge from a catchment area tributary to a reservoir is of the utmost importance in the design of a dam and a spillway. Various formulas, both empirical and theoretical, have been devised for expressing the volume of discharge; but no formula has yet been devised that is generally applicable to all of the conditions that occur in practice. They should be used with the utmost discretion, and only after a careful study of all the factors, such as topography, nature and depth of soil, vegetation, average temperature, humidity, etc. The following are a few of the formulas proposed:

Fanning's formula	$Q=200M^{\frac{5}{6}}$
Dredge's formula	$Q=1300\dfrac{M}{L^{\frac{2}{3}}}$
Col. Dicken's formula	$Q=CM^{\frac{3}{4}}$

in which Q is cubic feet per second yielded from the whole area; M is area of watershed, in square miles; L is length of watershed, in miles; and C is 200 in flat country, 250 in mixed country, 300 in hilly country, for a rainfall of 3.5 to 4 inches, or 300 to 350 for a 6-inch rainfall.

It is necessary to know the monthly and daily rates of run-off from a catchment area, as well as the mean annual rate of run-off, as these will affect the design of the spillway for a dam. The greatest floods will occur either on barren catchment areas with steep slopes, or wherever heavy snowfalls are followed by warm, melting rains. In some portions of the West, sudden flood discharges have been recorded of 30 second-feet per square mile of catchment area, where a few days previously the flow was at the rate of $\frac{1}{12}$ of a second-foot per square mile.

TABLE IV

Discharge and Run-Off from Catchment Areas of Important Streams in Arid Region of the United States

State	River Basin	Observing Station	Alti-tude (ft)	Drainage Area (sq mi)	Discharge: Maxi-mum (sec-ft)	Discharge: Mini-mum (sec-ft)	Discharge: Mean Annual (sec-ft)	Discharge: Total Annual (acre-ft)	Run-Off Depth: Max (in)	Run-Off Depth: Min (in)	Run-Off Depth: Mean Annual (in)	Run-Off Per Square Mile per Annum (s.c-ft)
Arizona	Salt	Arizona Dam		12,260	143,290	320	3,170	2,297,000			3 50	0 26
Arizona	Gila	Florence		17,834	102,566	0	500	400,000			0 5	0 06
California	Sacramento	Collinsville	0	26,187	160,000	5,050	37,630	26,000,000			16 32	1 20
California	Cosumnes	Live Oak	150	580			1,234	914,000			29 52	2 18
California	Tuolumne	Lagrange	290	1,500	22,900	30	2,685	1,960,000			22 56	1 64
California	San Joaquin	Herndon	295	1,637	19,960	60	3,074	2,220,000			25 44	1 84
California	Kern	Bakersfield	112	2,345	5,342	80	1,110	650,000			5 22	0 38
California	Kings	Red Mountain	1,810	1,775	22,732	145						1 30
California	San Gabriel	Azusa	614	222	1,765	0						0 30
California	Santa Anna	Warm Springs	43	188	580	18						0 40
Colorado	Arkansas	Canyon City	5,330	3,060	1,750	124	814	570,000	1 20	0 08	3 80	0 27
Colorado	Rio Grande	Del Norte	7,865	1,400	5,930	200	1,040	755,000	3 57	0 21	10 12	0 74
Colorado	South Platte	Denver	5,183	3,840	2,125	42						0 10
Colorado	Cache la Poudre	Fort Collins	4,984	1,060	5,060	32						0 32
Idaho	Snake	Idaho Falls	4,714	10,100	51,300	2,000	9,380	6,870,000	4 56	0 20	12 72	0 94
Idaho	Weiser	Weiser	2,125	1,670	17,940	10	1,211	877,000			9 85	1 00
Idaho	Teton	Wilford		967	5,270	425						1 00
Idaho	Boise	Boise	2,880	2,450	10,130	550						1 20
Montana	W Gallatin	Salesville	5,380	860	10,750	100	1,930	710,000	5 48	0 39	15 70	1 15
Montana	Madison	Red Bluff	5,020	2,420	8,613	910	975	1,430,000	2 67	0 55	12 60	0 93
Montana	Missouri	Craig	3,028	17,615	28,600	1,740	5,100	3,800,000	1 34	0 14	4 00	0 27
Montana	Yellowstone	Horr	5,120	2,700	15,500	285	2,880	2,180,000	4 17	0 12	15 12	1 11
Nevada	East Carson	Rodenbah		415	5,540	290	760	550,000	7 38	1 00	25 00	1 84
Nevada	Humboldt	Battle Mt	4,515	7,800	3,131	5						0 08
New Mexico	Rio Grande	Embudo	5,806	7,000	8,555	140	1,280	920,000	0.98	0 02	2 70	0 18
Oregon	Owyhee	Nyssa		9,875	18,000	32	2,114	1,536,000	1 53	0 02	2 90	0 21
Oregon	Umatilla	Gibbon		353	4 217	74						1 50
Texas	Rio Grande	El Paso	3,697	30,000	17,000	0	1,490	1,090,000	0 45		0 70	0 05
Utah	Bear	Colliston		6,000	10,590	540	2,210	1,600,000	1 52	0 06	5 00	0 37
Utah	Weber	Uinta	4,514	1,600	7,980	100	980	690,000	3 26	0 13	8 07	0 59
Utah	Provo	Provo	4,456	640	2,600	146	519	376,000	3 47	0 26	11 01	0 81
Utah	Sevier	Leamington	4,674	5,595	2,330	35	485	350,000	0 35	0 01	1 15	0 09
Utah	Odgen	Ogden	4,310	360	2,133	30						0 50
Utah	Green	Blake		38,200	68,800	610						0 25
Washington	Yakima	Kiona	499	5,230	30,000	612						0 98
Washington	Spokane	Spokane	1,886	4,005	32,875	1,972						2 20
Wyoming	Shoshone	Lovell		2,770	12 340	150						
Wyoming	Laramie	Uva	7,153	3,179	2,570	0						0 06
Wyoming	North Platte	Orin	4,704	14,820	19,160							0 15

Table IV derived from a series of observations over a period of years up to 1900, shows the discharge and the run-off from catchment areas of the more important streams of the arid region.

Compensation by Storage. Frequently, it becomes necessary to impound enough water to carry over a period of 2 or 3 years of minimum rainfall. A measurement of the drainage area having been obtained by surveys, a decision must be made as to the probable average, minimum, and maximum run-offs, both by year and by cycle of years. An estimate of the consumption of water must also be made, and from these figures a calculation of the storage to be provided may be made. If the minimum annual yield is equal to or greater than the consumption, storage will be required for only the dry season of one year of drought; if the minimum daily yield equals the maximum daily consumption, no storage will be required; if, however, the assumed consumption is nearly or quite equal to the mean yield, all of the surplus from the years of greatest rainfall must be stored and carried over until times of drought.

Evaporation and seepage from the reservoir must be considered and should be added to the consumption, in making the calculation for storage. A study of the run-off for a series of years at any location will give the approximate capacity required, that the consumption may not exceed the average yield.

The loss from evaporation is discussed later. There will be loss from seepage through the dam. The loss into the ground may usually be considered as additional storage; although for some reservoirs the seepage into the ground is lost, since the water reappears in springs too low to be used on the irrigated lands. The loss from seepage through a masonry dam should be small. The loss through an earthen embankment may be considerable, and the amount so lost will depend upon the character of the embankment, which should be so constructed that the daily seepage shall not exceed 10 gallons per square foot of vertical longitudinal section of embankment. With good materials and care in construction, the loss by seepage may be reduced to 5 gallons, or even as low as 3 gallons, per vertical square foot of embankment.

When irrigation is practiced, all of the water flowing in the streams is not available for storage, since much of it has already been appropriated by irrigators, and of course the quantity must be

deducted from the total water available. A large portion of the discharge occurs in winter when the streams are covered with ice, which renders it practically impossible to divert the water for storage, though it is available for such reservoirs as may be located on the main streams. As nearly all of the flow occurring in the irrigating season is appropriated, only the surplus and the flood water are available for storage.

LOSSES OF WATER

EVAPORATION

Factors Influencing Evaporation. If all precipitation upon a given area reached the adjacent stream as run-off, the amount of run-off would simply be the product of the total area of the watershed times the precipitation. Much of the precipitation, however, returns to the air by evaporation and transpiration. The rapidity with which water, snow, and ice are converted into vapor depends upon the relative temperature of these substances and the atmosphere, and upon the humidity of the air and the amount of motion in the latter.

Evaporation is greatest when the atmosphere is driest, when the water is warm, and when a brisk wind is blowing. It is least when the humidity is highest, the air quiet, and the temperature of the water low. In summertime the cool surfaces of deep waters may condense moisture from the atmosphere, and really gain in moisture when they are supposed to be losing water by evaporation. When, however, the reverse conditions exist in the atmosphere, and the winds are blowing briskly across the water, the resultant wave motion increases the agitation of the water, and the vapors from it escape freely into the unsaturated air which is constantly coming in contact with it and absorbing its vapors. Evaporation is therefore going on at a rate depending upon the temperature of the surface, and condensation may likewise be going on from the vapors which exist in the atmosphere; and the difference between these two factors represents the amount of water lost by evaporation or the amount gained by the water by condensation. Evaporation, therefore, should be greatest in amount in the desert regions of the South-

west, and the least in the high mountains; and it is known that, in the same latitude, evaporation differs greatly with the altitude.

Measurement of Evaporation. Different methods have been devised for measuring evaporation, none of which is wholly satisfactory. Most measurements of evaporation have been made from water surfaces by an evaporometer, the Piche evaporometer being the one most used in this country. Water surfaces form but a small proportion of the total area of most catchment basins; but the amount of evaporation from these can be ascertained with some degree of accuracy, while that from the earth and vegetation cannot be determined very accurately.

Evaporometers. *Results with Piche Evaporometer.* The Piche evaporometer and the Richards evaporation gage are thought to give inaccurate results, since they are affected by the temperature of the air only, which is seldom the same for any length of time as that of a near body of water. However, in 1888, a series of observations with a Piche evaporometer were made by Mr. T. Russell, of the United States Signal Service, to ascertain the amount of evaporation in the West; and the results obtained showed such slight discrepancies from the results obtained by other methods that they must be considered reasonably reliable. Observations were made with this instrument in wind velocities up to 30 miles per hour, from which it was discovered that with a velocity of 5 miles per hour the evaporation was 2.2 times that in quiet air; 10 miles per hour, 3.8 times; 15 miles per hour, 4.9 times; 20 miles per hour, 5.7 times; 25 miles per hour, 6.1 times; and 30 miles per hour, 6.3 times. Similarly to the precipitation in regions only a short distance apart, the evaporation may vary widely. For example, at Berkeley, California, in 1905, the evaporation was 41.6 inches, and at Tulare, California, it was 62.9 inches.

Floating-Pan Evaporometer. Another method of measuring evaporation from water surfaces is by measuring the actual loss from a pan containing water and floating in a lake or other body of water. Such a pan and scale are shown in Fig. 3. The pan is so placed that the contained water has as nearly as possible the same temperature and exposure as that of the body of water the evaporation of which is to be measured. This pan is of galvanized iron; 3 feet square and 10 inches deep, and is immersed in water and kept from

sinking by means of floats of wood or hollow metal. It should be placed in the body of water in such a position that the water in the pan will be exposed as nearly as possible to the average wind currents. This pan must be filled to within 3 or 4 inches of the top so that the rocking due to the waves produced by the wind shall not cause the water to slop over; and it should float with its rim several inches above the surface, so that the waves from this will not enter the pan. The device for measuring the evaporation consists of a small brass scale hung in the center of the pan. The graduations are on a series of inclined cross-bars so proportioned that the vertical heights are

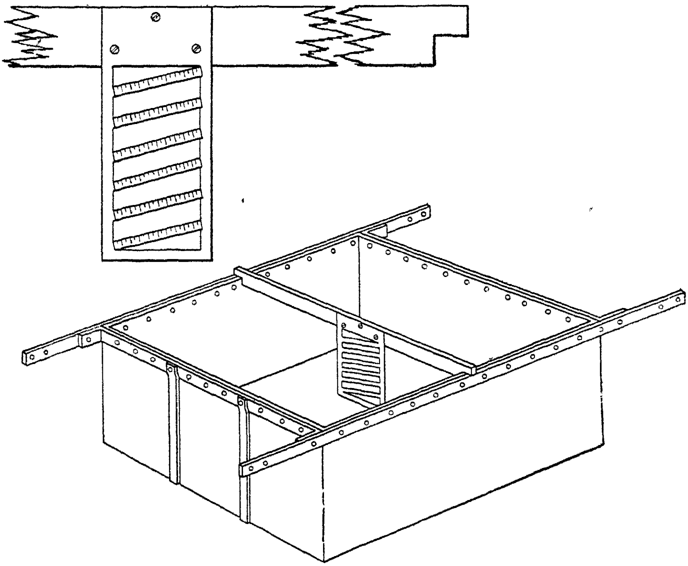


Fig 3 Floating Pan Evaporometer

exaggerated, thus permitting a rise or fall of 0.1 inch to cause the water surface to advance or retreat on the scale 0.3 inch. In this way, multiplying the vertical scale by 3, it is possible to read to 0.01 of an inch.

Amount of Evaporation. *Evaporation from Snow, Ice, and Water Surfaces.* From experiments conducted at the Boston Waterworks, the evaporation from snow was found to average 0.02 inch per day, and that from ice to average 0.06 inch per day (or $2\frac{1}{2}$ inches and 7 inches per season, respectively). In the arid regions of the West,

TABLE V

Evaporation from Water Surfaces in Various Parts of the United States

Location	Evaporation					
	Monthly (in)			Annual (in)		
	Max	Min	Mean	Max	Min	Mean
Massachusetts, Boston	7 50	0 66	3 29	43 63	34 05	39 20
California, Sweetwater	9 02	0 25	4 51	58 65	48 68	53 88
New York, Rochester	6 20	1 51	2 61	34 4	30 0	31 3
Middle Atlantic States				48 1	25 2	39 9
South Atlantic States				51 6	38 4	45 3
East Gulf States				56 6	45 4	50 6
West Gulf States				52 4	45 6	48 9
Ohio Valley and Tennessee				54 8	44 5	49 4
Lower Lake				38 6	32 9	35 8
Upper Lake				36 8	23 0	27 7
Upper Mississippi				52 2	28 1	38 8
Extreme Northwest				31 0	22 1	26 7
Arizona, Yuma						95 7
California, San Diego						37 5

the evaporation from snow will probably exceed this, especially on barren mountain tops exposed to the action of the wind and bright sunshine. Table V gives the amount of evaporation from water surfaces for various parts of the United States.

Evaporation from Soils. Still more important than the evaporation from water, is that from soils of different characters. That from vegetation is also important. Water once lost by evaporation from irrigated soil is practically lost permanently, and methods to prevent such loss should be employed. Evaporation from the soil depends upon the following: percentage of moisture contained in the soil; the general character of the soil, such as texture, composition, etc.; climatology, including temperature, humidity, wind movement, etc.; and the vegetation covering the ground. The moisture in the soil depends upon the rainfall or irrigation water applied, and upon the ability of the soil to absorb and retain the water reaching it.

In order to determine the amount of evaporation from soils under various conditions of their surfaces, the irrigation division of the United States Department of Agriculture, has carried on investigations on the effect of different depths of soil mulch and of cultivation. The total evaporation from soils receiving a 6-inch irri-

gation and free-water surfaces in 21 days in June, 1908, at different stations in the West was as follows: at Davis, California, free-water surface 8.27 inches; no mulch, 1.35 inches; 3-inch mulch, .32 inch; 6-inch mulch, .13 inch; 9-inch mulch, .03 inch: at Reno, Nevada, free-water surface 4.68 inches; no mulch, 1.41 inches; 3-inch mulch, .88 inch; 6-inch mulch, .36 inch; 9-inch mulch, .17 inch: at Wenatchee, Oregon, free-water surface 6.12 inch; no mulch, .86 inch; 3-inch mulch, .24 inch; 6-inch mulch, .13 inch; 9-inch mulch, .07 inch. The average evaporation for 21 days at five different stations

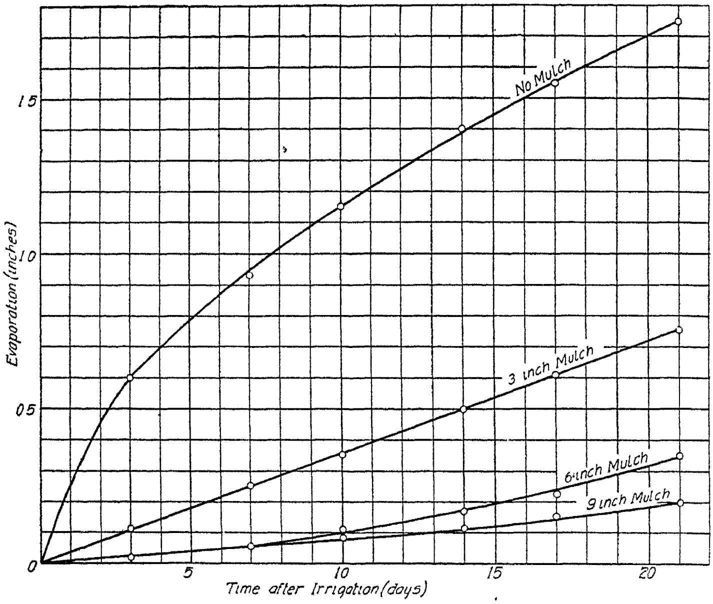


FIG. 4 Variation of Evaporation with Different Soil Mulchings

is given as follows: free-water surface 5.59 inches; no mulch, 1.75 inches; 3-inch mulch, .75 inch; 6-inch mulch, .34 inch; 9-inch mulch, .22 inch. The average evaporation for the different periods is shown graphically in Fig. 4.

The effect of cultivating the soil on evaporation is shown graphically by the diagram in Fig. 5, which is an average for a 28-day period at six stations throughout the arid West. The importance of early cultivation after irrigation is indicated by the steepness of the curves during the first few days after the water was applied.

In these experiments the first cultivation was given 3 days after irrigation and the second cultivation was given 14 days after irrigation.

Value of Water Storage. The value of water storage for irrigation in the West is realized chiefly during the dry season, May to August, inclusive. Little or no rain falls in the arid region during this interval over most of the area, and it is during these months that the evaporation from storage supplies is principally felt. In central California the average rainfall during these months is a trifle

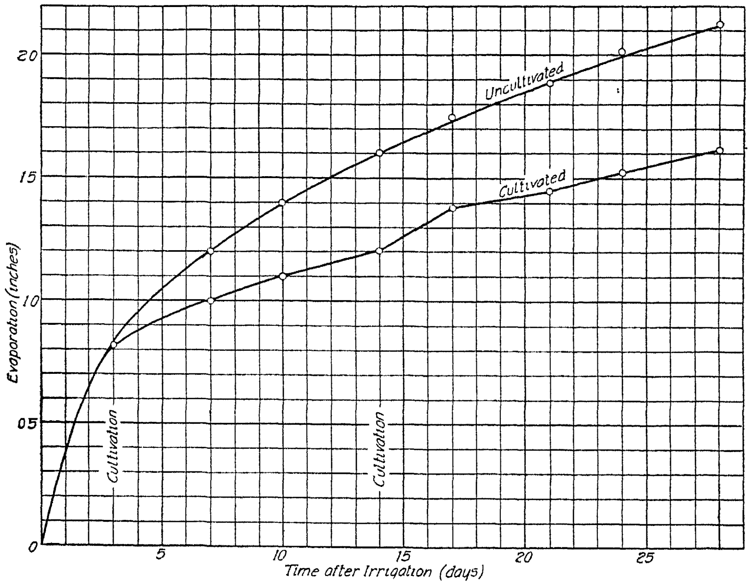


Fig 5 Variation of Evaporation with Soil Cultivation

less than 1 inch. The evaporation during the same interval is about 21 inches, causing a deficiency of 20 inches, due to evaporation. When the reservoirs are located at high altitudes in the mountains, the losses from evaporation are less than when located upon the hot lowlands.

In estimating the amount of water to be stored in a reservoir, allowance must be made for losses due to evaporation and seepage in the reservoirs and in conducting the water to the fields. This will seldom average below 25 per cent, and the amount of water stored in the reservoir must be large enough to supply this loss.

ABSORPTION

Percolation. In canals and storage reservoirs the percolation loss is large and varies according to the soil. The losses due to percolation alone are difficult to estimate, but, combined with evaporation, may vary from a very small amount to 100 per cent of the amount entering the canal. Actual gains have been noted in canals associated with drainage areas. The combined losses due to percolation and evaporation may be most conveniently considered under the head of *absorption*.

Losses by Absorption. In considering the losses of water due to absorption in canals and reservoirs, the nature of the bed must be taken into account. If the bottom is of sandy soil, the loss from evaporation is insignificant compared with that of percolation for most canals. If, however, the bottom is of clayey material, or if the canal or reservoir is old and the bottom protected by sediment, the loss from percolation will be limited and may not exceed that due to evaporation.

In new canals and reservoirs the losses due to absorption are greatest, and in a long line of canal may amount to from 40 to 60 per cent of the volume entering the head, and may be great enough to cover the whole canal 20 feet deep per day. In shorter canals the loss will be proportionately less, though rarely falling below 30 per cent. As the canal increases in age, the silt carried in suspension is gradually deposited upon the banks and bottom, filling up the interstices and decreasing the amount of percolation. In favorable soil, old canals varying in length from 30 to 40 miles may not lose more than 12 per cent, though the loss by main canals of average length may range from 15 to 40 per cent of the total volume entering the head, and the loss in laterals may be as much more.

To reduce the losses from percolation, it is recommended by Mr. J. S. Beresford, of India, that pulverized dry clay be thrown into canals near their headgates. This will be carried long distances and be deposited on the sides and bottom of the canal, forming a clay lining.

The losses by absorption may be greatly increased by giving the canal a bad cross-section, and in this feature of construction the attempt should be made to reduce the wetted perimeter and the surface of thé water exposed to the atmosphere.

Effects of Seepage. In considering the matter of seepage, it will be of interest to inquire into the effect of irrigation upon the subsurface water level. Before irrigation becomes universal, the subsurface water level may be so low that it will frequently be impossible to derive water from wells. The practice of irrigation, however, for a considerable time, seems to fill the soil with water so that the level of the subsurface water is raised and shallow wells often yield uniform and persistent supplies. It is stated that in Fresno, California, where the subsurface level was originally at a depth of from 60 to 80 feet below the surface, the seepage from the canals has raised this level so that wells 10 to 15 feet in depth now receive constant supplies. In "Irrigation Institutions", Mr. Mead says:

> Up to a certain limit, irrigation on the headwaters of a river is a benefit to the users of water below. About one-third of the water diverted returns to the stream as waste and seepage The waters diverted during the flood season, which return as seepage, come back slowly and help swell the stream when it is low and water is most needed The exact time of the return varies, of course, with the location of the lands irrigated and with the character of the soil, but in a general way the effect of the diversion of floods in irrigation is to equalize the flow of rivers They carry less water when high, and more water when low. Some rivers leaving the eastern slope of the Rocky Mountains, which formerly ran dry every year, now have a perennial flow; and on others, the period every year at which they become dry is traveling eastward rather than westward.
>
> Up to a certain limit, the storage of water also tends to equalize the flow of streams. Reservoirs are filled when there is an abundance, and the water is turned out when there is a scarcity. Hence the people who live along streams below where the stored water is used derive an indirect benefit from the increased seepage thereby created There is, however, a limit beyond which irrigation on any stream does not improve the supply of those living below If the irrigated valley is long enough, and the irrigated district broad enough, the ultimate absorption of the water supply is inevitable.

As illustrating the possible effects of seepage, the State Engineer of Colorado, during the years 1890 to 1893, inclusive, conducted examinations on the South Platte and Cache la Poudre rivers, with the object of determining the amount of seepage water returned to them. In 1893, on the South Platte River, in a distance of 397 miles, there was a gain of 573 second-feet over that found at the upper measuring station. In 1891 there was a gain of 300 per cent over the flow at the upper measuring station.

From experiments conducted in 1889 on the Cache la Poudre, it was found that the discharge at a point considerably down the

stream from the canyon was 214.7 second-feet, as against 127.6 second-feet at the canyon—and this, after supplying 15 canals and without receiving additional supplies from drainage. These results were borne out by experiments conducted through succeeding years.

Experiments of a similar nature conducted elsewhere point to the same results. The amount of water returned by seepage will depend upon the soil, the nature of the underlying strata, the amount and direction of slope, and the extent of the drainage area above and tributary to the streams. It is probable that in many cases the amount of seepage water returned to the streams will be practically nothing; and in designing reservoirs it will be safer to assume that the gains by seepage water will be offset by the losses due to percolation, evaporation, etc.

QUANTITY OF WATER REQUIRED

DEFINITION OF TERMS

Duty of Water. It will first be necessary to define the term *duty of water* as applied to irrigation, in order to determine, at least approximately, the amount of water necessary to supply a given area for a specific purpose. *Duty of water* may be defined as *the ratio between a given quantity of water and the area of the land which it will irrigate.* On the duty of water depends the financial success of every irrigation enterprise, as it involves the dimensions and cost of construction of canals and reservoirs, and the feasibility of furnishing a sufficient supply of water at a reasonable cost.

Units of Measure for Water Duty and Flow. In ordinary hydraulic problems the standard unit is the *cubic foot.* In irrigation problems, however, where large volumes of water are to be considered, the cubic foot is too small a unit and the *acre-foot* is the standard adopted by irrigation engineers. An acre-foot is the amount of water which will cover an acre of land one foot in depth—or, it is equivalent to 43,560 cubic feet.

In considering irrigation streams, as rivers or canals, the volume of flow must be coupled with a time factor, both representing the rate of flow. As in other hydraulic problems, the time unit usually employed by irrigation engineers is the *second,* and the unit of measure of flowing water is the *cubic foot per second*—or the *second-*

foot, as it is termed for brevity. Thus the number of second-feet flowing in a canal is the number of cubic feet passing a given section in a second of time.

Another unit still generally employed in the West is the *miner's inch*. This differs widely in different localities, and is generally defined by state statute. In California one second-foot of water is supposed to be equal to 50 miner's inches. Since the miner's inch is so variable and the amount actually obtained by its use in measuring water is questionable, the use of the cubic foot per second is advised and is generally being adopted by irrigation engineers and others.

The period of time during one season, determined between the first watering and the completion of the last watering, is the *irrigating period*. This is usually divided into several *service periods* —that is, the times during which water is allowed to flow on the land for any given watering. The irrigation period in most of the western states, extends from about April 15 to August 15. The service period, or the duration of one watering, is generally from 12 to 24 hours, according to soil, crop, and water supply; and the number of waterings making up the irrigation period varies between 2 and 5, depending upon the soil, climate, and crop.

VARIATION IN DUTY OF WATER

Affecting Conditions. The duty of water may be expressed by the number of acres of land which a second-foot of water will irrigate, by the number of acre-feet of water required to irrigate an acre of land, or in terms of the total volume of water used during the season. It is also sometimes expressed in terms of the expenditure of water per linear mile of the canal, when the location of the canal has been previously determined. On account of the losses of water by evaporation, seepage, etc., while flowing through the canal, care should be taken to state whether the duty is reckoned upon the basis of the water entering the canal or upon the amount of water applied to the land. In a long line of canal, the losses may be a large portion of the total amount entering the canal, and the relative duties would vary accordingly.

The duty of water in various portions of the West is variable. Investigations show that it is rapidly rising; for, as land is irrigated

TABLE VI
Water Distribution, 1913—U. S. Reclamation Service

State	Project or Unit	Delivery to Farms (acre-feet) Jan	Feb	Mar	Apr	May	June	July
Arizona	Salt River ( )	9,297	24,583	31,989	59,121	64,829	64,458	61,633
Ariz.-Calif.	Yuma	1,454	5,995	7,486	8,378	8,504	10,418	12,022
California	Orland		17	2,076	2,384	4,417	4,109	4,616
Colorado	Uncompahgre					27,865	32,817	30,866
Idaho	Boise — Upper (')				1,356	26,967	38,589	24,616
	Boise — Lower (')				2,019	15,885	16,279	15,779
	Boise — Total				3,375	42,852	54,868	40,395
Idaho	Minidoka — Gravity					31,585	50,855	54,254
	Minidoka — Pumping					7,784	10,862	18,438
	Minidoka — Total							
Montana	Flathead					37	2,556	2,477
Montana	Fort Peck						350	450
Montana	Huntley					1,440	4,059	10,080
Montana	Milk River				27	226	307	1,426
Montana	Sun River					161	3,832	4,240
Mont.-No. Dak.	Lower Yellowstone						5,438	3,549
Neb.-Wyo.	North Platte					9,527	33,856	33,072
Nevada	Truckee-Carson			8,002	22,567	29,691	11,950	11,825
New Mexico	Carlsbad			839	7,013	6,112	2,481	6,324
New Mexico	Hondo	30			102		356	
New Mex.-Tex.	Rio Grande		5,315	23,072	23,883	29,534	25,788	6,153
North Dakota	No. Dakota Pumping						1,312	479
Oregon	Umatilla			100	5,450	7,950	8,550	6,550
Ore.-Calif.	Klamath					4,990	8,147	4,285
South Dakota	Belle Fourche					554	12,851	22,631
Washington	Okanogan					1,539	2,259	3,808
Washington	Yakima — Sunnyside				18,677	35,059	31,284	37,276
	Yakima — Tieton					7,670	8,316	11,024
	Yakima — Total							
Wyoming	Shoshone				263	7,685	7,949	10,162
Summary		10,781	35,910	73,564	151,240	331,750	404,047	401,442

(1) Data is for calendar year, 1913, except that on Salt River project the data is for corresponding agricultural year, Oct 1912—Sept 1913
(2) October, 1912, commencing agricultural year
(3) November, 1912 (4) December, 1912
(5) Exceeds figures given in other tables by 5367 acres, mainly in towns
(6) Above Deer Flat reservoir (7) From Deer Flat reservoir

TABLE VI—Continued
Water Distribution, 1913—U. S. Reclamation Service

Delivery to Farms (acre-feet)						Irrigated Area	Delivery per Acre Irrigated (acre-feet)				
Aug	Sept	Oct	Nov	Dec	Total	(acres)	1913	1912	1911	1910	1909
60,176	55,260	(2)27,463	(3)25,174	(4)16,005	499,988	(5)168,679	2 97	3 94	3 53	3 6	4 2
10,009	8,947	5,785	3,078	3,335	85,411	19,607	4 36	4 6	5 43	3 10	4 85
2,233					19,852	6,61	3 00	3 97	3 9	3 4	
30,922	23,532	14,054			160,056	31,428	5 09	4 81	5 44	6 25	4 5
12,777	1,653	1,695			107,653	50,383	2 13				
14,861	7,115	625			72,563	25,882	2 85				
27,638	8,768	2,320			180,216	(8) 76,265	2 38	1 93	1 79	1 67	2 39
55,032	36,309	7,787			(9)235,822	43,873	5 38				
17,091	10,039	244			64,458	32,084	2 01				
727	217				6,014	4,631	1 30	1 98	1 99	4 53	
					800	410	1 9				
6,982	1,557				24,118	15,798	1 53	1 5	1 88	2 03	2 0
215	20	129			2,350	2,545	0 92	0 82	1 28		
1,602	1,352				11,187	7,419	1 51	1 71	1 65	2 3	
701	415	147			10,250	7,660	1 34	1 19	1 41	1 44	0 72
40,595	24,439				141,489	56,829	2 49	2 25	4 26	3.93	3 17
9,035	8,920	4,985			106,975	43,075	2 48	2 50	4 46	4 65	5 01
5,648	2,937	889	801		33,044	14,260	2 32	2 9	2 2	2 4	2 3
	230	82			800	932	0 86	1 3	0 9	0 9	
1,396	1,000	4,463			(10)120,604	27,723	(10)4 34	(10)5 4	(10)6 0	(10)4 4	(10)6 6
496					2,287	1,739	1 31	0 86	1 23	1 63	1 53
8,150	5,550				42,250	4,994	8 45	8 2	9 7	10 2	8 0
4,040	699				22,161	18,928	1 17	1 13	1 23	0 88	1 30
7,668	3,645				47,349	32,881	1 44	1 10	1 64	1 95	1 66
4,277	110				11,993	7,700	1 57	1 24	1 27	2 46	2 5
38,431	28,123	16,090			204,940	58,300	3 51	3 06	3 08	3 29	3 48
10,435	5,094				42,539	18,750	2 27	2 27	1 91	1 73	
9,323	3,682	1,373			40,437	19,423	2 08	1 66	2 20	2 05	2 54
357,835	234,510	86,221	29,053	19,340	2,135,693	(11)722,550	2 96				

(8) Exceeds figures given in other tables by 18,000 acres of New York Canal Co 's lands.
(9) Delivered to sublaterals (10) At lateral head gates.
(11) Exceeds total given in other tables by 23,367 acres, as indicated in foot notes (5) and (S).

through a series of years, it becomes more saturated, and as the level of the ground water rises, the amount of water necessary for the production of crops diminishes. The cultivation of the soil causes it to require less water, and the adoption of more careful methods in designing and constructing distributaries, and care and experience in handling water, increase its duty. In the same state, and even in the same neighborhood, the duty will vary with the crop, soil, and altitude.

Experiments have shown that a good, heavy rain amounting to $5\frac{1}{2}$ inches soaks into the earth to a depth of from 16 to 18 inches or more. If this amount of water were applied three times in the season, it would be equivalent to a total depth of $16\frac{1}{2}$ inches to the crop. An average depth of 3 inches of water on the surface is sufficient to water an average soil thoroughly. The number of waterings required in a season depends upon the crop, soil, climate, etc. For the average crop and soil in a southern arid region four or more irrigations may be required, while farther to the North, and where the rainfall comes mostly during the growing season, one or two irrigations may be ample. The actual amount of water required to mature the crop depends upon the crop, the soil, the climatic conditions, the cultivations, the methods of irrigation, and other factors.

It has been the tendency for the irrigators to apply more water than necessary, and to make irrigation take the place of cultivation, with the result that many thousands of acres of some of the most valuable lands of the arid sections have been ruined by seepage water and alkali. In many places the water-logged lands have been reclaimed by drainage. In planning a new project the question of drainage should also be considered, or, better still, the amount of water applied should be limited to beneficial and economical use.

Water Distribution and Crop Returns. Extensive investigations have been conducted to determine the actual amounts of water applied to the crops by the farmers in the irrigated sections, and other investigations on the economical duty of water have also been made. Table VI gives the water distribution on the U. S. Reclamation projects for 1913 as given in the *Thirteenth Annual Report* of the Reclamation Service. The amount delivered

TABLE VII

Acreage Cropped and Value of Crops, 1913, U. S. Reclamation Service Projects

State	Project or Unit	Acreage Cropped	Crop Value	
			Total	Per Acre Cropped
Arizona	Salt River	161,642	$4,552,879	$28 17
Ariz -Calif	Yuma	16,726	610,228	36 48
California	Orland	5,987	203,949	34 07
Colorado	Uncompahgre	30,366	99,153	32 77
Idaho	Boise	50,865	830,314	16 32
Idaho	Minidoka { Gravity	36,879	593,105	16 08
	Minidoka { Pumping	29,362	543,347	18 51
	Minidoka { Total	66,241	1,136,452	17 00
Montana	Blackfeet			
Montana	Flathead	4,579	53,846	11 76
Montana	Fort Peck	410	1,960	4 78
Montana	Huntley	15,798	464,697	29 35
Montana	Milk River	2,459	24,004	9 76
Montana	Sun River	6,807	105,564	15 51
Mont -No. Dak.	Low. Yellowstone	7,410	101,587	13 71
Neb.-Wyo.	North Platte	54,306	786,621	14 40
Nevada	Truckee-Carson	42,943	555,007	12 92
New Mexico	Carlsbad	12,195	257,274	21 10
New Mexico	Hondo	808	14,236	17 62
New Mex -Tex.	Rio Grande	26,720	679,271	25 40
North Dakota	No. Dak Pump { Buford-Trenton, Williston	1,686	38,888	23 06
Oregon	Umatilla	3,033	84,078	27 72
Ore -Calif	Klamath	18,928	288,189	15 22
South Dakota	Belle Fourche	32,568	355,380	10 91
Washington	Okanogan	2,736	86,438	31 59
Washington	Yakima { Sunnyside	46,230	2,820,786	61 00
	Yakima { Tieton	12,595	422,950	33 60
	Yakima { Total	58,825	3,243,736	55 14
Wyoming	Shoshone	18,178	262,464	14 44
Summation, 1913		642,216	$15,732,215	$24 50
Summation, 1912		539,934	$13,825,369	$25 60

to the farms for each acre irrigated varied from 0.86 acre-foot to 8.45 acre-feet with an average for all projects of 2.96 acre-feet.

Table VII taken from the same report gives the value of the crop returns. By comparing the tables it is noted that the crop returns with the application of a depth of 0.86 foot were $17.62 per acre, and with the application of 8.45 feet they were $27.72 per acre; with 1.9 feet the returns were $4.78 per acre, the lowest; with 3.51 feet the returns were $61.00 per acre, the highest; and, on one project, with the application of 2.97 feet, which is about the

average, the returns were $28.87; while on another project, with the application of 3 feet, the returns were $34.07.

There are many factors besides the amount of water applied that affect the crop returns, but the tables given are of valuable assistance in estimating the amount of water required to irrigate a certain tract of land, or in estimating the amount of land that can be irrigated with a given quantity of water.

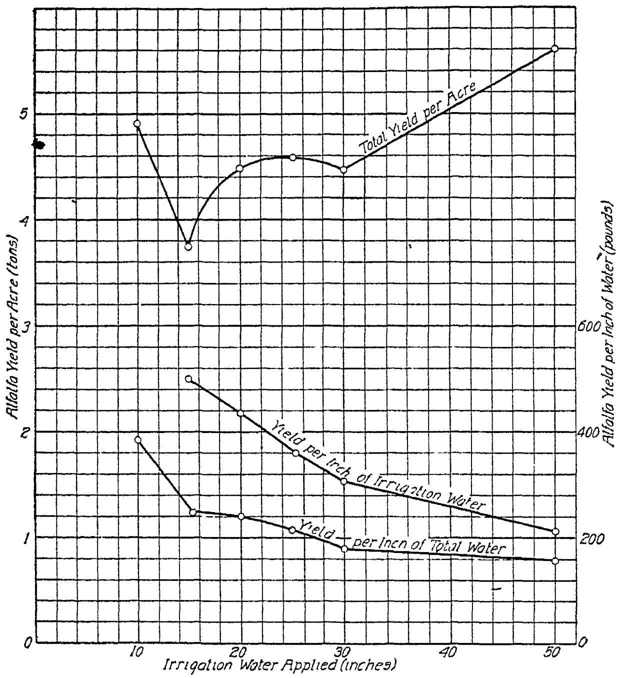


Fig 6 Variation in Alfalfa Production with Amount of Irrigation Water

Several years' work has been done on the economical duty of water at the Utah Experiment Station, and the curves of Fig. 6 indicate the variation in yield of alfalfa with the amount of water applied. Nearly all crops experimented with showed the same characteristic curves. After a certain point has been reached, the increase in the crop returns is very little for the amount of additional water applied.

Ratio of Actual to Theoretical Duty. In every irrigated area only a small percentage of the total area commanded is irrigated

in any one season. Some of the land is occupied by woods, farmhouses, or villages. Some is occupied by pasture lands that receive sufficient moisture by seepage from adjoining irrigated fields, and some lands are allowed to lie idle during a season. From estimates made of the area under cultivation in wild portions of the West, it is found that if water is provided for 500 acres out of every section, it will be sufficient to supply all of the demands of the cultivators. It will be seen, therefore, that the actual duty of water, when estimated on large areas, is at least 20 per cent greater than the theoretical duty per acre.

IRRIGATION SYSTEMS

PRELIMINARY CONSIDERATIONS

Land To Be Irrigated. In examining into the feasibility of a proposed irrigation project, the first consideration is the land

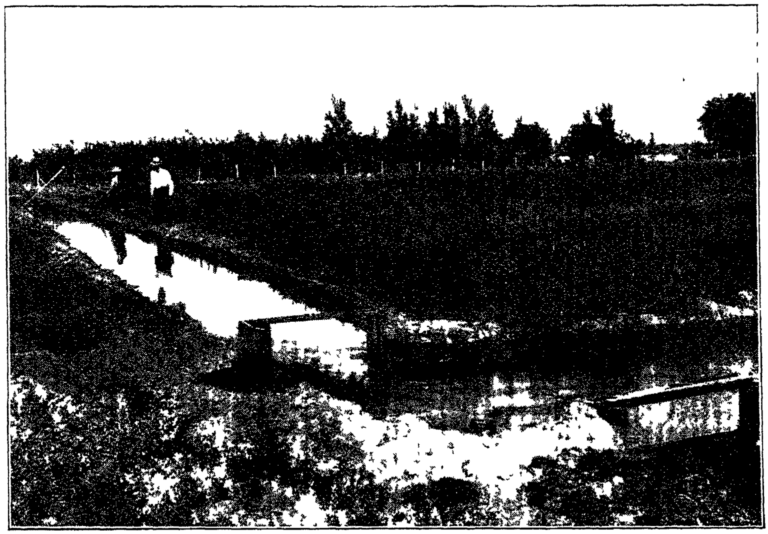


Fig 7 Rio Grande Irrigation Project, Los Cruces, New Mexico

to be irrigated. The area of this must be considered, its proximity to markets, the nature of the soil, the climate, and the value and character of the crops. The value and ownership of the land must also be considered, for, unless the proposed irrigation results in

increased quantity and improved quality of the crops, the value of the land will not be enhanced and the project will result in failure. In Fig. 7 is shown an experimental irrigation plot.

To determine the area and configuration of the land under consideration, a topographical survey will be necessary, and a plot should be made to as large a scale as possible, upon which the contours should be drawn at intervals of from 5 to 10 feet.

Determining the Source of Supply. Having determined all the matters relative to the area, quantity, and value of the land, and the necessity of supplying water for irrigation, the next step is to determine the source of supply and its location relative to the lands. This supply may be taken from an adjacent perennial stream, or it may be necessary to transport it from a neighboring watershed, or, again, it may be necessary to conserve in reservoirs the intermittent flow of minor streams. Or the water may be derived from subterranean sources, from which it may flow under pressure, or be lifted by pumps. The relation of the water supply to the land, the extent and value of the latter, and the volume and permanency of the former, are the vital points to be determined in the preliminary investigations of an irrigation project.

If the source of supply is a perennial stream, it must be examined as to its velocity and quantity of flow during high and low stages. A topographical survey of the watershed may be necessary to determine its area. The minimum and maximum depth of rainfall, and the probable run-off should be determined. Minor streams of more or less intermittent flow may be examined as to the feasibility of bringing them together and impounding their supplies for use over dry seasons. Subterranean supplies can be examined by driving test wells to determine their source, quantity of flow, and permanency.

CLASSIFICATION OF WORKS

Having determined the source of water supply and its relation to the irrigable lands, the next step is the design of the irrigation works. These may be divided into two great classes: (1) gravity works and (2) pumping, or lift irrigation.

The sources of supply are perennial streams, intermittent streams, artesian wells, or the storage of perennial, intermittent,

or flood waters, for gravity works; and, for lift irrigation, may be all kinds of wells, canals, storage works, or flowing streams.

The conditions necessary to the development of an irrigation canal are: *first,* that it shall be carried at as high a level as possible when additional irrigable land can be covered, in order to command as great an area as possible; *second,* it should be fed by some source of supply that will maintain the water at a constant level; *third,* it should have such a slope and velocity as to prevent as far as possible the deposition of sediment and the growth of weeds, and at the same time have such a velocity that the cross-section may be a minimum for a given discharge, provided that the scouring action is not so marked as to endanger the canal itself.

Climate, geology, and topography are the determining factors as to the particular class of works adapted to a particular region.

GRAVITY WORKS

Gravity works cover all those forms of irrigation by which the water is conducted to the land with the aid of gravity or natural flow, and includes perennial canals, periodic and intermittent canals, artesian-water supplies, inundation canals, storage works, and subsurface or ground-water supplies.

Perennial Canals. Canals, deriving their supplies from perennial streams or storage reservoirs, may be divided into the two following classes, according to the location of the headworks: highline canals; and low-service or deltaic canals.

Highline Canals. Highline canals are usually designed to irrigate lands of limited area, and are given the best possible slope in order that they may be as high as possible, generally, to command the maximum area of land. In such canals the headworks are usually located high up on the streams, frequently in rocky canyons where the first portions of the line may encounter heavy and expensive rock work.

Low-Service Canals. Low-service canals are constructed where the majority of the lands are situated in low-lying and extensive valleys, and when the location of the head of the canal depends, not so much on its being at a relatively high altitude and commanding a great area, as upon the suitability of the site for purposes of diversion. Deltaic canals have been constructed in India and

Egypt at the deltas of some of the great rivers. They are low-lying canals of relatively large cross-section and low velocity of flow.

Intermittent and Periodic Canals. Intermittent and periodic canals are usually of small dimensions, commanding relatively small areas of land, and are generally employed by individual farmers to eke out a supply for which the annual precipitation is nearly sufficient.

Storage Works. Storage works may be of almost any capacity, depending upon the nature of the project, the source of supply, and the area of the land to be irrigated. They may be built in connection with perennial canals, and are especially necessary in connection with intermittent canals, artesian wells, and subsurface or ground-water supplies.

Inundation Canals. Inundation canals are used almost exclusively in India and Egypt, and derive their supply from streams the beds of which are at a relatively high altitude compared with the surrounding country. It is only necessary, when the water in the river is high, to make a cut through its banks and permit it to flow out into the canals, which distribute it over the surrounding country. They rarely require any permanent headworks to control the entrance of the water to the canal.

PUMPING, OR LIFT IRRIGATION

Under pumping or lift irrigation are included those forms of irrigation in which the water does not reach the land by natural flow, but is pumped by means of animal power, windmills, steam power, electric motors, water wheels, elevators, gas engines, or hydraulic rams. Frequently large volumes of water are found situated at such low levels that the water cannot be distributed by gravity, and must be raised by pumps or other lifting devices to be distributed directly over the lands, or stored in reservoirs for distribution by gravity.

As irrigation is practiced, the subsurface soil becomes saturated, the ground-water level rises, and much of the water delivered by gravity may be pumped up and re-employed for irrigation, increasing thereby the duty of the water supply. In this country rapid progress has been made in recent years along lines of irrigation pumping, and thousands of acres of land are amenable to cultivation by this

method, that cannot be irrigated by gravity supplies. It is estimated that $9,000,000 have already been spent on 250,000 horsepower to pump water to irrigate 260,000 acres of land.

Of all types of farming, that by irrigation pumping possibly requires the most skill to make a success of it. Before the installation of a pumping plant is made, the problem should be carefully

Fig. 8 Double-Wheel Windmill and Storage Reservoir for Irrigation Purposes

investigated. The questions to be decided upon are crops to be raised, markets for the crops, water supply, lift, method of pumping, kind of power and cost of power, and such others as have any bearing on the success of the project.

Power Considerations. *Windmills.* Windmills have been extensively used by the individual farmer in portions of the West for raising water for irrigation. Fig. 8 shows a windmill recently

developed for irrigation pumping. Fig. 9 shows windmills in and around Deming, New Mexico. In this latter instance, the water is

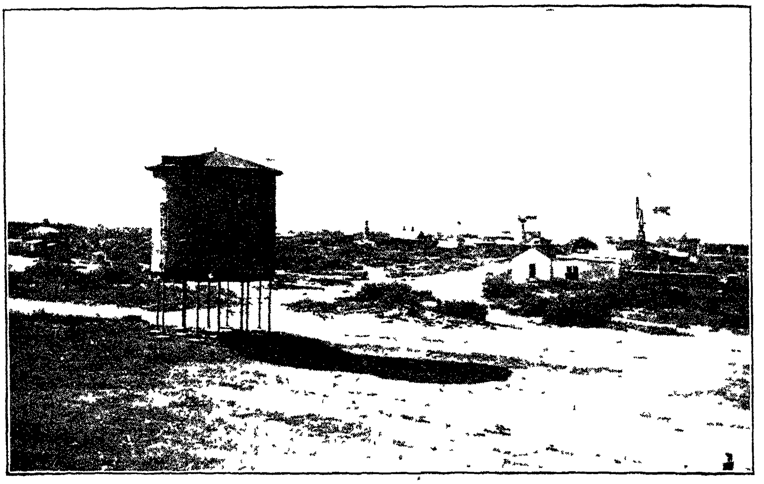


Fig 9 Windmills for Pumping, and Storage Tank for Irrigation Water, Deming, New Mexico

reached at a depth of about 60 feet from the surface and is pumped to a storage tank, from which it is distributed over the land as

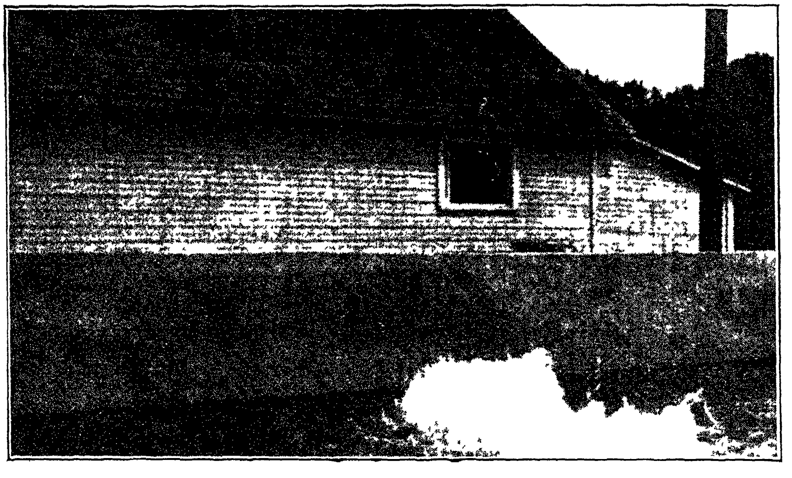


Fig 10 Private Reservoir and Pumping Plant in Western Kansas

Water is pumped from well by fuel-oil engine into storage reservoir to obtain sufficient head for irrigation

required. Windmills, however, are rather expensive to maintain, requiring constant care, and the storage tanks need continual attention.

Electric and Gas Power. If the water supply is obtained from wells, the farmer's individual plant may be preferable to a central station. This will save main and lateral canal losses and reduce the draw-down in the wells. The rapid development of the internal-combustion engine during recent years and the improvement in the design of pumps have both been factors influencing the development of pumping irrigation. In places, a low grade of fuel oil costing but a few cents a gallon can be used for the engine fuel. The actual amount used in pumping varies between wide limits under different conditions of installation, but with a well-designed plant the amount should not be much in excess of half a gallon to raise 1 acre-foot of water 1 foot. In other places cheap electrical power will help develop irrigation pumping. Fig. 10 shows a view of a modern individual pumping plant where a fuel-oil engine is used for power.

Types of Pumps. The pumps commonly used in irrigation pumping are: *centrifugal* pumps, both horizontal and vertical, with open and enclosed impellers; *turbine* pumps, and *deep-well plunger* pumps, used for deep wells with small bore; *water elevators*, and to a small extent, *pumping engines*, *pulsometers*, and *hydraulic rams*.

PARTS OF A CANAL SYSTEM

Principal Divisions. Taking up the design of gravity systems, a great perennial canal consists of the following parts: (1) main canal; (2) head and regulating works; (3) control and drainage works; and (4) distributaries and laterals.

The principal units of this system are the *headworks*, *main canals*, and *distributaries*. Between different canal systems, the greatest points of difference are found in the headworks and in the first few miles of diversion line, where numerous difficulties are frequently encountered, calling for variations in the form and construction of drainage works and canal banks.

The *headworks* usually consist of the diversion weir, with its scouring sluices, of the head-regulating gates at the canal entrance, and of the head or first-escape gates. The *control works* consist of regulating gates at the head of the branch canals, and of escapes on the line of the main and branch canals. The *drainage works* consist of inlet or drainage dams, flumes or aqueducts, inverted siphons, and drainage cuts. In addition to these works, there are usually

constructed falls and rapids for neutralizing the slope of the country, and tunnels, cuttings, and embankments. Modules or some form of measuring boxes or weirs are necessary for the measurement of the discharge.

The headworks of a canal are usually located high up on the supplying stream in order to command a sufficient area and to tap the stream where the water is clear and contains the least amount of silt. By so locating the headworks, it is usually possible to reach the watersheds with the shortest possible diversion line. The disadvantages of this class of location are serious, since the canal line is intersected by hillside drainage, entailing serious difficulties in construction, and as the adjacent slopes of the country are heavy, much expensive hillside cutting is required.

Diversion lines are those portions of a canal system that are required to bring the water to the neighborhood of the irrigable lands. Since they do not of themselves command any irrigable land, the endeavor should always be to reduce the length of diversion lines to a minimum, so that the canal shall command irrigable land and derive revenue at the earliest possible point in its course.

MAIN CANAL

Alignment. The alignment of the canal should be such that the canal will reach the highest part of the irrigable lands with the least length of line and at a minimum expense of construction. The line of the canal should follow the highest line of the irrigable lands, skirting the surrounding foothills, and passing down the summit of the watershed dividing the various streams. The best alignment can be determined upon only after careful preliminary and location surveys have been made of the country involved. These should include a complete topographic survey, and a plot of it to as large a scale as possible, the contour lines being spaced at vertical intervals of from 5 to 10 feet. On such a plot it is possible to lay down, with a close degree of approximation, the final position of the canal line. Such a plot frequently renders possible an improved location, saving many miles of canal by the discovery of some low divide or some place in which a short, deep cut or tunnel will save a long roundabout location. The final location may now be made in the field, with the aid, perhaps, of a few short trial lines.

A direct or straight course is the most economical, as it gives the greatest freedom of flow with the least erosion of the banks. It also greatly diminishes the cost of construction, as well as the losses by seepage and evaporation consequent on the increased length of a less direct location. It is an error in alignment to adhere too closely to grade lines following the general contour of the country. By the insertion of an occasional drop, it may be possible to obtain a more desirable location and to diminish the cost of construction by avoiding a natural obstacle.

The careless location of curves is a serious error in alignment, as the insertion of sharp bends results in the destruction of the banks, or requires that they be paved to protect them from erosion. Curvature diminishes the delivering capacity of a canal. As the cross-section becomes smaller or the velocity increases, the radius of curvature should be correspondingly increased. To maintain the discharge of a canal constant throughout its length, either its cross-section or its grade should be increased in proportion to the sharpness of the curve.

Such obstacles as streams, gullies, and unfavorable or low-lying soil or rocky barriers are frequently encountered in canal alignment, and the best method of passing these must be carefully studied. It may be cheaper to carry the canal around these obstructions; or it may be better to cross them at once by aqueducts, flumes, or inverted siphons, or to cut or tunnel through the ridges. Careful study should be made of each case, and estimates made of the cost, not only of first construction, but of ultimate maintenance. In crossing swamps or sandy bottom lands, it may be cheaper, because of the losses due to evaporation and seepage, to carry the canal in an artificial channel.

If water is abundant, it may be the less expensive, on hillside work, simply to build the canal with an embankment on its lower side, permitting the water to flood back on the upper side according to the slope of the country. On account of the growing scarcity of water, however, any method that would be wasteful of water should not be adopted without considerable deliberation. The relative cost of building a hillside canal, wholly in excavation or partly in embankment should be considered. If the hillside is steep and rocky, the advisability of tunneling, of building a masonry

retaining wall on the lower side of the canal, or of carrying the water in an aqueduct or flume, will have to be considered.

In finally locating an expensive work, borings and trial pits should be made—the former by means of a light steel rod, and the latter by simple excavation—in order to discover the nature of the material to be encountered. In making the final survey of a canal, it is well to place at convenient intervals permanent benchmarks of stone or other suitable material. The establishment of these along the side of the canal in some safe place, will give convenient datum points to which levels may be referred wherever it may be necessary to make repairs or new branch lines.

Slope and Cross-Section. Slope and cross-section are dependent the one upon the other. Having determined the discharge required, the carrying capacity for this discharge may be obtained by increasing the slope and the consequent velocity and diminishing the cross-sectional area, or by increasing the cross-sectional area and diminishing the velocity.

The proper relation between cross-section and slope requires the exercise of careful judgment. In order to reduce the deposition of silt and the growth of weeds to the minimum, it is desirable to give the water as high a velocity as the material will stand. This may result, however, in bringing the water to too low an elevation to command the area of land desired. Too great a cross-sectional area may result in excessive cost, if the material is in rock or for any other reason is difficult to remove. Other things being equal, the correct relation of slope to cross-section is that in which the velocity will neither be too great nor too small, and yet the amount of material to be removed will be reduced to a minimum.

When the fall will permit, the slope of the bed of the main canal should be less than that of the branches; and the latter should be less than that of the bed of the distributaries and laterals, the object being to secure a nearly uniform velocity throughout the system, so that sedimentary matter in suspension will not be deposited until the irrigable lands are reached.

Limiting Velocities. In order that the proper slope may be chosen—one that will prevent deposit on the one hand and at the same time not erode the banks—it is necessary to know the limiting velocities for different materials. In a light, sandy soil, surface

velocities of from 2.3 to 2.4 feet per second, or mean velocities of 1.85 to 1.93 feet per second, give the most satisfactory results. Velocities of from 2 to 3 feet per second are ordinarily sufficient to prevent the growth of weeds and the deposition of matter in suspension; and other things being equal, this velocity should be maintained whenever possible. Ordinary soil and firm, sandy loam permit velocities of from 3 to 3½ feet per second; while in firm gravel, rock, or hardpan, the velocity may be as high as 5 or 7 feet per second. Brickwork or heavy dry-laid paving or rubble will not stand velocities higher than 15 feet per second;

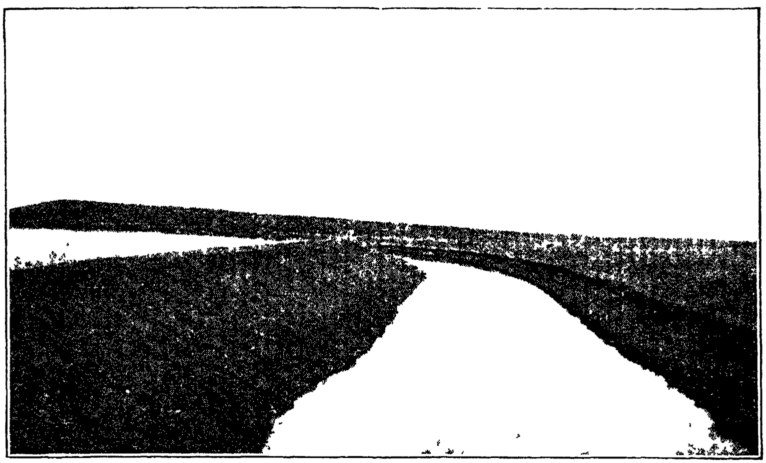


Fig 11. Irrigation Canal Showing Berm

and only the most substantial form of masonry construction is capable of resisting still higher velocities.

The grade required to produce these velocities is dependent chiefly upon the cross-sectional area of the channel. Much higher grades are required in canals of small cross-sectional area than in large, in order to produce the same velocity. The velocity required being known, the grade may be ascertained from Kutter's or some similar formula. (See page 79.) Or, on the other hand, if the grade is limited, the resulting velocity may be determined. In large canals of 60-foot bed width and upwards, and in sandy or light soil, grades as low as 6 inches to the mile produce as high velocities as the material will stand. In firmer soil this grade may be increased

to 12 to 18 inches to the mile, whereas smaller channels will permit of slopes of from 2 to 5 feet or more per mile, according to the material and dimensions of the channel.

Design of Cross-Section. Theoretically, the most economical form of cross-section of channel is one with vertical sides and a depth equal to one-half of the bottom width; but of course this form is applicable only to the firmest rock. The best trapezoidal form is one in which the width of the water surface is double the bottom

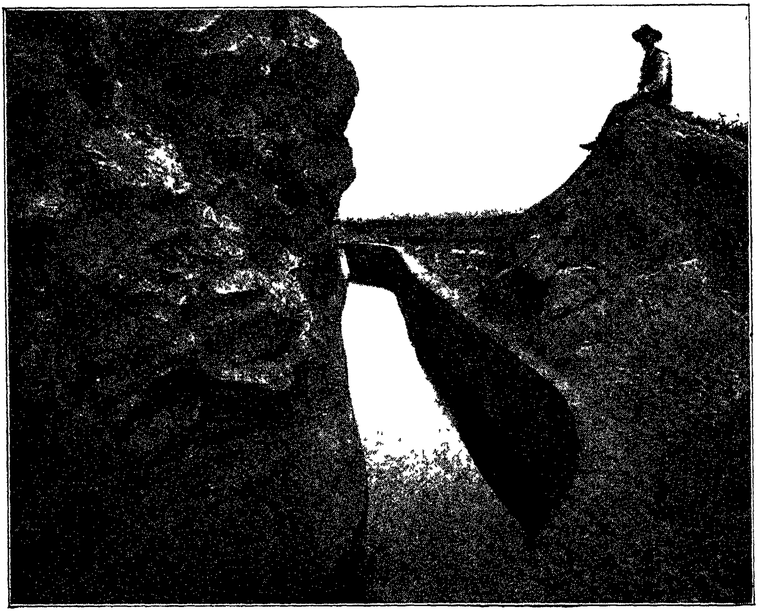


Fig 12 Portion of Upper Main Canal at Pogue Flats, Okanogan Project, Washington
The Okanogan project by storage of the waters of the northern Salmon River provides for the irrigation of 8000 acres of fruit lands near Okanogan

width. The side slopes above water level should be as steep as the material will permit.

The particular form of cross-section will depend upon the nature of the material and the topography. The greater the depth, other things being equal, the greater will be the velocity. Fig. 11 is a view showing a canal designed to carry a large or a small amount of water. The object of the berm is to decrease the width for small quantities and to increase the width for large quantities. Fig. 12 shows a concrete irrigation ditch, on a curve with almost vertical

sides, and Fig. 13 shows a straight section of a concrete ditch with a trapezoidal section

Very large canals, such as some of those in India, have been given a proportion of depth to width similar to that of great rivers. This proportion has been found to be most nearly attained when the bed width is made from 15 to 16 times the depth. In hillside excavation, the greater the proportion of depth to width, the less will be the cost of construction, and in rock and heavy material it is

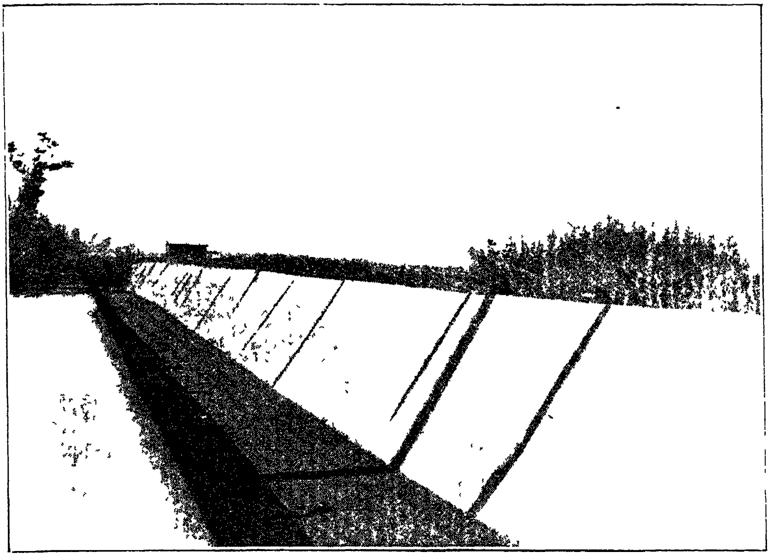


Fig 13 Concrete Canal with Sloping Sides for Carrying Pumped Water in Kansas

desirable to make the bottom width not greater than from 2 to 3 times the depth.

The cross-section of a canal may be so designed that the water may be wholly in excavation, wholly in embankment, or partly in both. The conditions that govern the choice of one of these three forms, are dependent upon the alignment and grade of the canal, and upon the character of the soil. It may be desirable at times to keep the canal wholly in cut, provided the topography and consequent location will permit of it. For if the material of which the banks are constructed is light and porous, the water may filter through and stand in stagnant pools on the surface, causing

unnecessary waste as well as unsanitary conditions. If the material is impervious and will form good, firm banks, it may be well to keep the canal in embankment where possible, although this may necessitate the expense of borrowing material. To reduce the cost of construction, it is desirable, where the location will permit, to keep a canal about half in cut and half in embankment, thus reducing to the minimum the amount of material to be handled.

Most main canals follow the slope of the country in grade contours running around hillside or mountain slopes. In such cases it is necessary to build an embankment on one side only, when the cutting will be entirely on the upper side. If there is a gentle slope on the upper side, and consequently an embankment on that side, it is desirable to run drainage channels at intervals from this embankment to prevent the water making its way through it to the canal. These drainage channels may be taken through the embankment into the canal, or may be led away to some natural watercourse.

In large canals it is always desirable to have a roadbed on at least one bank, and the width of this will determine the top width of the bank. The inner surfaces of the canal are usually made smooth and even; while the top is also made smooth, with a slight inclination outward to throw drainage water away from the canal. The inner slopes of the banks vary in soil from 1.1 to 4:1, according to the character of the material. In firm, clayey gravel or hardpan, slopes of 1:1 may be constructed. In ordinary, firm soil mixed with gravel, or in coarse, loamy gravel, slopes of $1\frac{1}{2}$ 1 are necessary. In other soils, a slope of 2:1 will be required; and light, sandy soils will require a slope of 4:1.

Embankment. The top width of the canal bank is generally from 4 to 10 feet, according to the material and depth, and whether or not the canal is in embankment. If there is to be no roadway on the top of the embankment, and the surface of the water does not rise more than a foot or so above the foot of the embankment, a top width of 4 feet is sufficient. Where the depth of water on the embankment is greater, the top width should be 5 or 6 feet, and in light soil it should be 10 feet.

It may be necessary to build a puddle wall in the embankment, or to make a puddle facing on the inner slope in case the material is particularly pervious. The same effect is obtained by sodding

or by causing grass to grow on the bank. During the construction of the work, the material may be compacted by putting it in place in layers and thoroughly rolling.

The carrying capacity of a canal should be so calculated that the surface of the water where in cut shall not reach within less than 1 foot of the ground surface. In fill, the surface of the water should not come within less than $1\frac{1}{2}$ feet of the top of the bank, while in extreme cases it may be unsafe to allow the water to approach closer than 2 feet of the top of the bank.

HEADWORKS

The term *weir*, as distinguished from *dam*, refers to any structure for the impounding or diversion of water, over which flood waters may flow without endangering the structure. Thus weirs

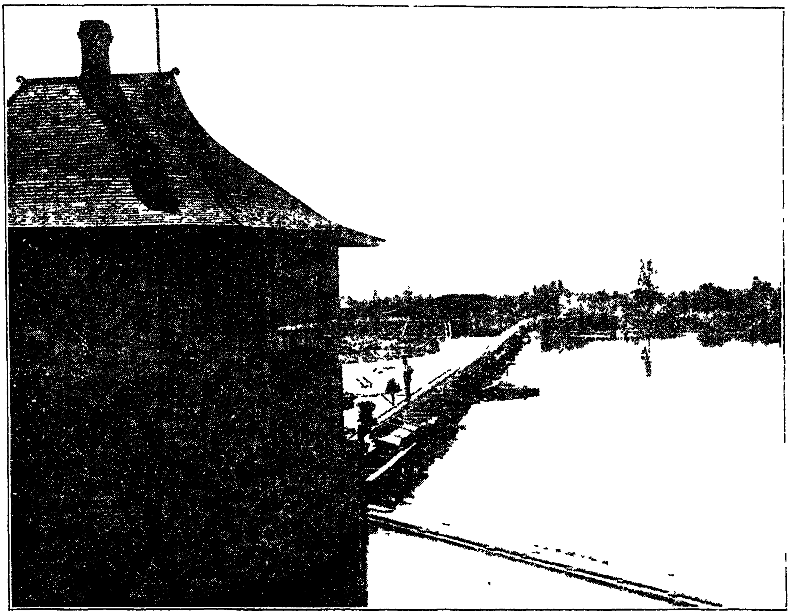


Fig 11 Gate House and Dam, Sunnyside Canal Heading

are usually built across streams, near the heads of canals, for the diversion of the water of the stream into the canal, while the surplus water flows over the weir and passes down the stream. In some cases, however, dams are built over which it would be unsafe to

allow water to pass, and the surplus is then discharged through a spillway constructed around one end of the dam.

The headworks of a canal, as has been noted, are usually located at the highest possible point where the stream emerges from the hills. At such a point, the topography of the country and the elevation of the stream make it possible to conduct the water to the irrigable lands with the shortest diversion line and the least loss of

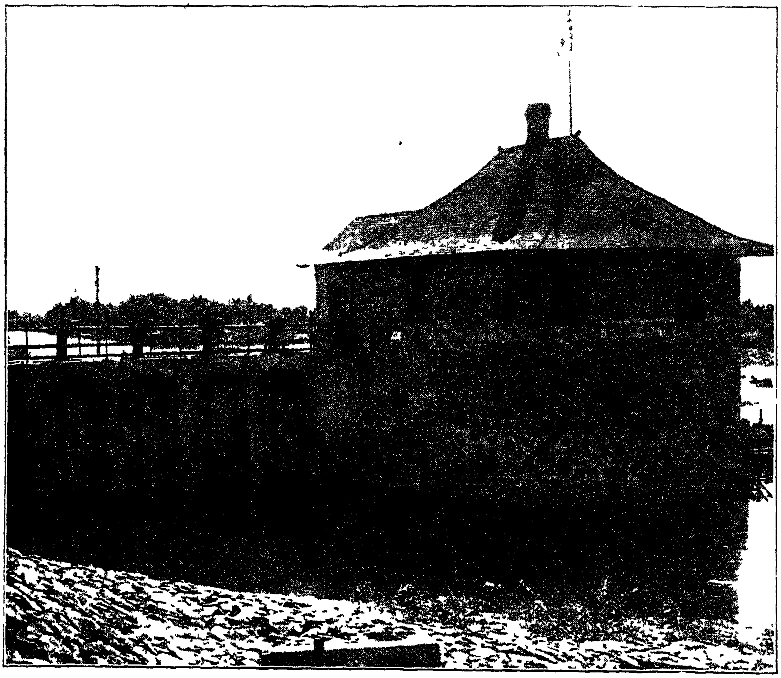


Fig 15 Gate House and Intake Gates, Upstream Face, Sunnyside Canal

A view of the Yakima project, Washington This project calls for the reclamation of 160,000 acres of land in the Yakima Valley Flood water is saved by storage in Lakes Keechelus, Karhees, and Clealum, in the Cascade Range The private headworks of the Sunnyside Canal Company were purchased by the Government, and have been reconstructed of reinforced concrete, the canal being sufficiently enlarged to care for the necessary irrigation waters

head. At such a location, also, the width of the channel of the stream is usually contracted, permitting of a reduction in the length of the weir and in the consequent cost of construction. At times it may be necessary to locate the headworks at a point where the distance between banks is greater, in order that the depth of water on the weir during flood discharge may not be excessive, thus endangering its stability. Obviously the points to be kept in mind in the

location and design of headworks are: permanency; economic first cost; cost of maintenance; and the command of the maximum area of irrigable lands at the least cost of diversion canals. These requirements may at times be conflicting; and complete surveys, supplemented by careful study of the problem in all of its details, are necessary to a satisfactory final conclusion.

The headworks of a canal consist of. (1) diversion weir or dam; (2) scouring sluices; (3) regulator at head of canal, for its control (see Figs. 14 and 15); and (4) an escape for relief of canal below.

Diversion Weirs

Principal Types of Construction. Diversion weirs or dams, as the term indicates, are structures built for the purpose of backing up or impounding a body of water in order that it may be diverted into the proper channels to reach the irrigable lands. They may be classified as follows, according to the construction material of the superstructure: (1) brush and bowlder weirs; (2) earthen dams; (3) masonry dams and weirs; (4) loose rock-fill dams; (5) wooden dams and weirs; and (6) iron or steel dams and weirs. These materials are also used in various combinations. The best kind of dam to build depends upon its size, the character of the foundation, the topography of the site, the degree of imperviousness required, materials available, and the cost.

Brush and Bowlder Dams. Brush and bowlder dams require practically no engineering skill in their design and are of the simplest construction. They are usually low barriers thrown up across a stream to raise the water only a few feet. Conditions are sometimes met under which it is advisable to build such a dam. These dams are constructed by driving stakes across the stream bed and attaching to them bundles of twigs, usually willow and cotton wood, and weighting the twigs down with rock and gravel. The brush end is laid upstream. Alternate layers of brush and rock and gravel are laid in the dam until the height may reach 3 or 4 feet. Fig. 16 is a view of a brush dam in the Tongue River in southern Montana.

Earthen Dams. The earthen embankment is one of the most common forms of dam. It can be built on a variety of foundations and is usually cheap to build and, when properly designed and constructed, it furnishes a safe and reliable structure. Where water

must be passed over the top of the dam, some material other than earth must be used. Ample provision for passing the maximum flood waters must be made. The foundation for an earthen dam should contain an impervious stratum at a moderate depth and the material must be sufficiently compact to support the load. Compact clay or hardpan makes the best foundation for earthen dams. Solid rock without fissures also forms a good foundation. The top width of an earthen dam will depend upon the material, the height, and whether it contains a roadway. Solid, well-compacted clay

Fig 16 Brush Dam on Tongue River, Montana

should be used near the upper face of the dam; when coarse pervious materials must be used, they should be placed near the downstream face.

Core Walls. Core walls of impervious materials are sometimes necessary in an earthen dam near the upstream face. A core wall of puddled clay will vary in thickness for 4 to 8 feet at the top and about one-third the depth of the water at the bottom. Cores of masonry are of various thicknesses. In a dam constructed of good material, they may be only a foot or two thick, but generally they are from 2 to 4 feet thick at the top with a batter on both sides of $\frac{1}{2}$ to $\frac{3}{4}$ inch to the foot. Below the ground level the sides are vertical.

Riprap is usually employed on the inner face of the dam to prevent erosion by wave action. The top of the dam should be from 2 to 7 feet above high-water line, depending on the material, exposure to winds, and frost action. The inner slope will vary from 2 horizontal to 1 vertical, to 4 horizontal to 1 vertical, and on the lower side a slope of 2:1 may be used. On high embankments berms are placed 30 to 40 feet apart vertically.

Masonry Dams. *Materials.* For permanency in weirs, only steel and masonry should be used in the construction. Masonry weirs may be built either of plain concrete throughout, of concrete reinforced with steel, of uncoursed rubble masonry laid in cement mortar, of ashlar masonry, of brick masonry, or of combinations of these. Or they may be built of loose rubble maintained in place by masonry walls.

Classification. As regards the superstructure, masonry weirs may be classified according to the method of discharge, as follows. (1) simple weirs of moderate height and overflow, with a clear fall to the bed of the stream; (2) simple weirs with clear fall to an artificial apron; (3) weirs with rollerway on lower face; (4) weirs of heavy cross-section curved on the face to break the fall of the water; and (5) weirs with clear fall to a water cushion.

Foundations. Very firm foundations are required for masonry dams; the higher the dam, the firmer must be the foundation. Masonry dams 20 feet or 30 feet high have been constructed on firm earth, but the high dams must be founded on a foundation that will not be subject to settlement, or cracks will form. Low dams may be constructed upon heavy-timber foundations resting upon piles, or a solid hardpan or gravel foundation. For such dams the greatest care must be exercised to prevent undermining of the foundation.

Close sheet piling should be carried entirely across the upper end of the foundation, placed in a trench dug down well into hardpan or clay, and puddle well rammed into the trench on both sides of the sheeting, which should also be carried 2 or 3 feet higher than the foundation, and the space between it and the masonry filled with concrete or puddle, Fig. 17. The foundation should be carried downstream for a distance at least equal to the height of the dam to act as an apron to prevent undermining by the falling water. At

the end of the apron should be placed sheet piling similar to that already described.

General Structural Considerations. Theoretical considerations act as a guide in the design of masonry dams, the three requirements for safety being stability against sliding on its foundation, stability against overturning, and maximum stresses within safe limits. The top width of masonry dams varies from 4 or 5 feet for low dams, to 15 or 20 feet for very high dams, and the height of crest above the water line varies from 2 or 3 feet to 8 or 10 feet.

Masonry is practically unpliable, and so an unyielding foundation is necessary if there are to be no unknown and unprovided-for stresses in the structure caused by a yielding and insecure foundation. All loose or decomposed rock of the foundation should be

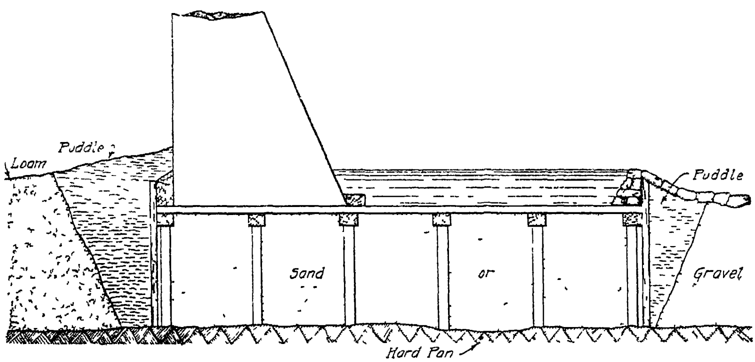


Fig 17 Sheet Piling in Weir Construction

removed; and if the foundation rock has a smooth top surface, a shallow trench should be excavated into it the entire length of the dam, or the bed should be cut in steps to prevent leakage and the sliding of the dam on its bed. Every square foot of the foundation should be thoroughly cleaned and washed and covered with a thick bed of mortar just before the masonry is laid upon it, and the spaces between the dam and the sides of the excavation should be cleaned out and filled with concrete. Seams in the rock should be filled with cement, to prevent the escape of the water behind the dam. Springs should be diverted or sealed up, to avoid risking the stability of the structure.

The ends of the dam should be carried to bedrock if possible, and treated in the same manner as the bottom. If rock does not

extend up to the level of the crest of the dam at its ends, they should be carried some distance into the banks, to prevent leakage around them, or tight masonry river walls should be carried for some distance upstream from the dam. The banks for a short distance downstream, if of earth, must be protected from wash by similar walls.

In building a masonry dam, care should be taken to break courses at every joint, the masonry being uncoursed rubble, except the faces, which should be broken ashlar. Every stone and its bed must be perfectly clean and have a damp surface when it is laid;

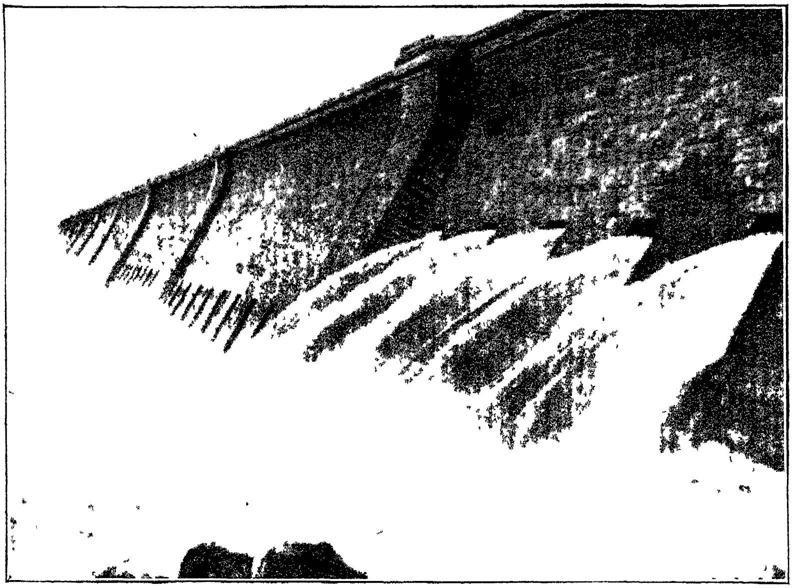


Fig 18 Assuan Masonry Dam, Nile River, Egypt, before Being Heightened

and all spaces must be absolutely filled with mortar or fine concrete. If a tight dam is to be built, these precautions must be rigidly observed. In constructing the dam, the work, so far as possible, should be carried up so that the finished masonry is approximately horizontal. All stone should be set by derrick; and no dressing of stones should be done on the wall, since that would be likely to disturb or jar the masonry already set. Careful attention to details will result in a dam perfectly water-tight under all practicable heads. Fig. 18 is an example of a masonry dam.

Concrete has been used in the construction of a number of dams, and the use of that material for this class of work is increasing

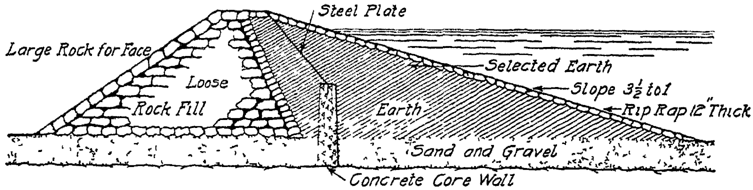


Fig 19 Section of Avalon Dam, Carlsbad Project, New Mexico

rapidly. Concrete reinforced with steel has been used in the erection of several notable structures, resulting in marked economy of cost.

The design of masonry dams and weirs will not be taken up here, as the matter is fully treated in the article on "Dams and Weirs".

Loose Rock-Fill Dams. In many places where it is desired to build a dam, rock is found in abundance and this is sometimes an economical material to form the body of the dam. Loose rock must be used with some other material such as timber, earth, or steel, in order to make the dam water-tight. Fig. 19 shows the cross-section of a rock-fill dam on the Pecos River in New Mexico. The maximum height is 52 feet and the top width is 14 feet.

Wooden Dams and Weirs. Timber-crib dams have been fre-

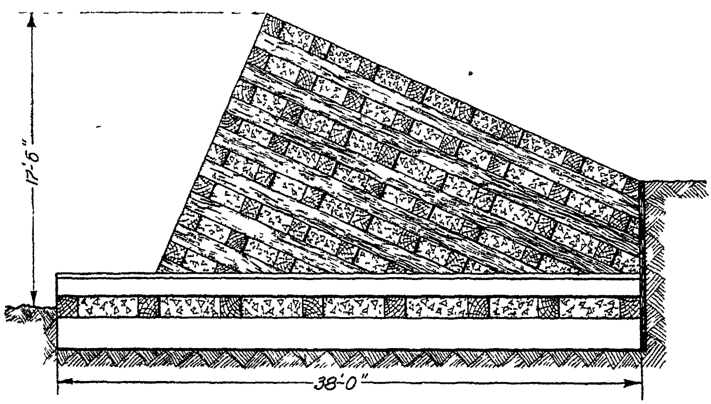


Fig. 20. Cross-Section of Bear River Weir

quently built, and, where properly constructed and the timber is not exposed to frequent wetting and drying, they give good service

Fig. 20 is the cross-section of the Bear River weir which is timber crib.

Wooden crib and rock weirs are generally built when the bed and banks of the river are of heavy gravel and bowlders or of solid rock; and they may be employed for diversions of greater height than is possible with open weirs. Crib weirs consist of a framework of heavy logs, drift-bolted together, and fitted with broken stones and rocks to keep them in place. The structures may be founded by sinking a number of cribs, one on top of the other, to a considerable depth in the ground, or they may be bolted to the solid rock. They may consist of separate cribs built side by side across the stream and fastened firmly together, or they may be built as one continuous weir.

Iron or Steel Dams. Dams of this sort have been built. One at Ash Fork, Arizona, 70 feet high and 184 feet long on top, is used for city water supply. It consists of a series of triangular steel bents, resting on concrete foundations. The face consists of steel plates $\frac{3}{8}$ inch thick, 8 feet $10\frac{5}{8}$ inches wide, and 8 feet long, riveted to 20-inch, 65-pound I-beams.

Open Weirs and Closed Weirs. Weirs are also classed as *open weirs* and as *closed weirs*. An open weir consists of a series of piers of wood, iron, or masonry, set at regular intervals across the bed of the stream, and resting on a masonry or wooden floor constructed flush with the river bed and protected from scouring by curtain walls constructed up and down stream. The piers are fitted with flashboards or movable weirs so constructed that the afflux height of the river may be controlled. The distance between piers varies from 3 feet to 10 feet. In a river subjected to sudden floods, the gates are constructed to drop automatically when the flow overtops them.

A closed weir consists of an apron built upon a substantial foundation and carried across the entire width of the stream, flush with the level of its bed, and protected from erosion by curtain walls up and down stream. The superstructure may consist of a solid wall, or may consist in part of upright piers, the interstices being closed by some temporary construction. This latter portion of the weir is called a scouring sluice. In a closed weir the barrier is solid nearly the entire width of the channel, the flood waters passing over the crest. Such weirs have usually a short open space

—the scouring sluice—in front of the regulator, the object of which is to produce a swift current past the regulator opening, to prevent the deposit of silt at that point.

Scouring Sluices. Scouring or undersluices are built in the bottom of every well-designed dam or weir, and at the end immediately adjacent to the regulator head. Their function is to remove by erosive action of the water any sediment that may be deposited in front of the regulator. (See Fig. 21.)

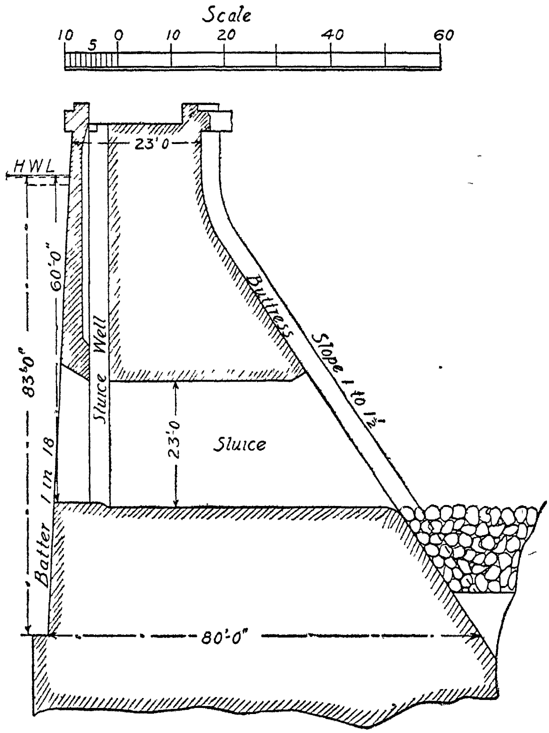


Fig 21 Section of Assuan Dam on River Nile
(See Fig 18 for Photographic View)

In case the flow in the stream is sufficient to warrant it, the sluices are kept constantly open, preventing the deposit of silt by the constant motion of the water in front of the regulating head. If the flow of water is not sufficient to permit of leaving the sluices constantly open, they should be opened during flood and high waters, creating a swift current which is effective in removing the silt deposited during slack water.

The scouring effect of sluices constructed in the body of the weir is produced by two classes of structures open scouring sluices and undersluices.

Open Scouring Sluice The open scouring sluice is practically identical with the open weir, as the latter consists of scouring sluices built across the entire width of the channel. When the weir forms a solid barrier across the channel and is open for only a short portion of its length adjacent to the canal head, the latter is called a *scouring sluice*. The waterway of a scouring sluice is open the entire height of the weir from crest to bed of stream, and consists of foundation, floorway, and superstructure. The floor must be deep and well constructed, and carried for a short distance up and down stream upon either side of the axis of the weir. On this floor are built piers, grooved for the insertion of planks or gates that may be opened or closed at will.

Undersluice. These sluices are more generally constructed where the weir is of considerable height and the amount of silt deposited is small. The opening does not extend as high as the crest of the weir, nor does the sill of the sluiceway necessarily reach to the level of the stream bed It is necessary, however, that the sill be as low as the sill of the regulator head.

CONTROL WORKS

Regulators. *Relative Location.* A diversion weir retards the flow of the stream and raises its level to such a height as to enable it to pass into the head of the canal. The regulator controls the flow of water, admitting it to the canal when necessary, and at other times causing it to pass on down the stream over or through the weir. The regulator should be so located that the water held back by the weir will pass rapidly, and with the least loss of head, through it and into the canal. To accomplish this successfully, the canal head should be placed immediately adjacent to the weir and continuous with it.

The weir should not be so located and aligned as to cross the stream diagonally, either toward or from the regulator head. In the former case, it tends to force the water against the regulator, causing unnecessary scour at that point and undue pressure upon the head. In the latter case, the reverse-action would take place,

and it would not be effective in directing the water into the canal. The best alignment for the weir is to have it cross the stream at right angles to the line of the regulator. This gives a clear, even scour past the regulating gates and keeps them clear of silt, at the same time furnishing the required amount of water.

The regulator should not be placed at a distance from the end of the weir, as thereby dead water, in which deposits of silt occur, is

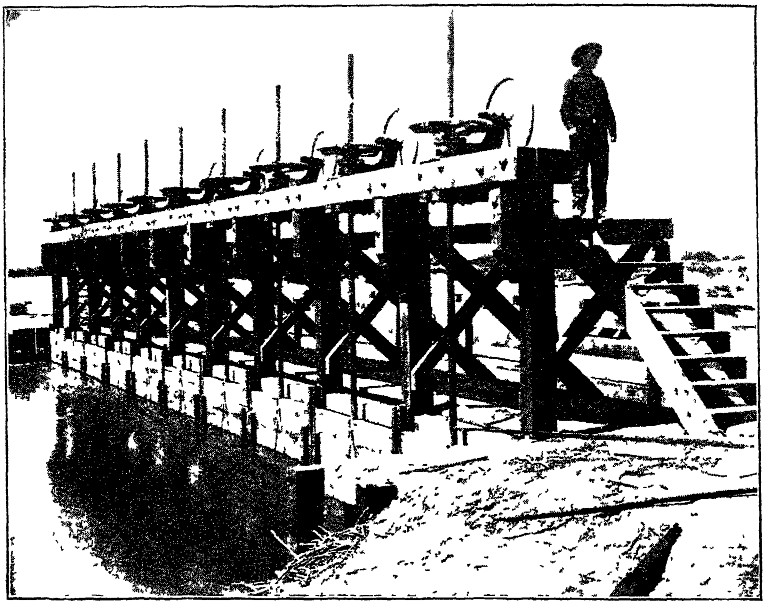


Fig 22 Gates at Sharp's Heading
At this point, seven miles east of Calexico, Mexico, the Alamo River drops from the Imperial Canal

created between the weir and the regulator, blocking the entrance to the canal and diminishing the available supply.

Types. The type of regulator employed depends upon the character of the foundation and the permanency desired. Regulators may be classified according to the design of the gate and its method of operation. With any type of foundation, varying degrees of permanency may be given to the superstructure, and various methods employed for operating the gates.

Regulators may be classified as follows: (1) wooden gates in timber framing; (2) wooden gates in masonry and iron framing; and (3) iron gates in masonry and iron framing.

Further classification, according to the method of operating the gates, is: (a) flashboard gates; (b) gates raised by hand lever; (c) gates raised by chain and windlass; (d) gates raised by screw gearing; and (e) hydraulic lift gates.

Simple flashboard gates can be used only when the pressure upon them is low. When the gates are under great pressure, the opening is generally closed by a simple sliding gate that may be raised by hand lever or windlass, Fig. 22. Under high pressures a double series of gates may be employed, one above the other, each separately raised by hand lever or windlass.

Arrangement. The regulator should be so designed that the amount of water admitted to the canal may be perfectly controlled at any stage of the stream. To this end, the gates should have such dimensions that they can be quickly opened or closed as desired When the canal is large and its width considerable, the regulator should be divided into several openings, each closed by a separate and independent gate.

The channel of the regulator way should consist of a flooring of timber or masonry, to protect the bottom from the erosive action of the water; and the side walls or wings should be of the same construction, to protect the banks. The various openings will be separated by piers of wood, iron, or masonry; and the amount of obstruction offered to the channel should be the minimum, so that the width of the regulator head will be as small as possible for the desired amount of opening.

For convenience in operating, the regulator should be surmounted by arches of masonry or a wooden flooring, so as to provide an overhead bridge from which the gates may be raised or lowered. The height of the regulating gates and the height of the gates surmounting them must exceed the height of the weir crest by the amount of the greatest afflux height which the flood water may attain, so that the flood flow may not top the regulator and destroy the canal. The regulator must be firmly and substantially built to withstand the pressure of floods, and a drift fender should be built immediately in front of, or at a short distance in front of, the gates, to protect them from floating logs, débris, etc.

The regulator head should be placed as close to the end of the weir as possible, to prevent the deposit of silt at that point. It is

sometimes necessary, however, to set the regulator back in the canal a short distance, owing to the nature of the banks and the consequent difficulties of construction. Under such circumstances the escape should be placed in front of and adjacent to the regulator, to relieve it of undue pressure.

Escapes. *Functions.* In order to maintain complete control over the water in a canal, provision should be made for disposal of any excess flow that may arise from sudden and excessive rains or floods, or of any water not required for irrigation. This is affected

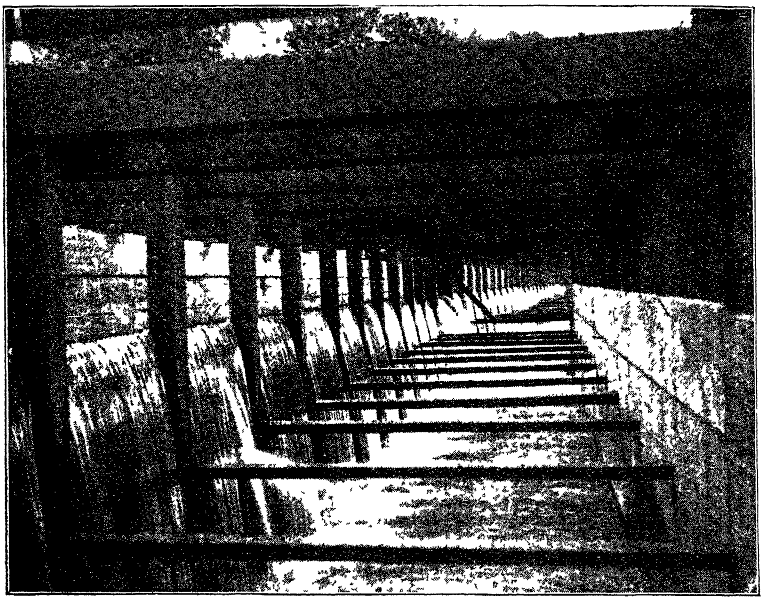


Fig 23. Interior of Automatic Spillway for Control of Water Supply in Canal, Yakima Project, Washington

by means of *escapes*, or, as they are more commonly called, *waste-ways* or *spillways*. These are usually short cuts from the canal to some natural drainage channel or stream into which the excess water may be wasted without fear of damage, Figs. 23 and 24. Wasteways may also be utilized for flushing the canal, preventing the deposit of silt, or scouring it out.

Location. Opening the distributaries relieves the main canal, and opening the escapes in turn relieves the distributaries. Escapes should be provided at intervals along the entire canal line, the length of each interval depending upon the topography of the country,

the danger from floods on inlet drainage, and the dimensions of the canal. The first or main escape should be located at a distance from the regulator not exceeding half a mile, so that in case of accident to the canal the water may be drawn off. This main escape may also be used as a flushing gate for the prevention and removal of silt deposits. The slope of the canal between its head

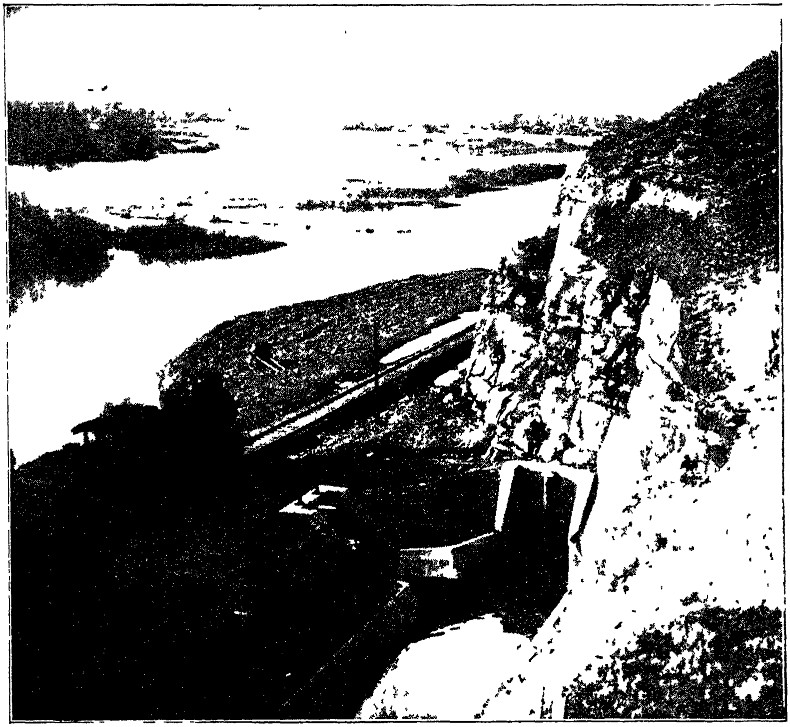


Fig 24 General View of Wasteway and Portal of Tunnel No 3 on Huntley Main Canal, and Passing Northern Pacific Train

The Huntley project, Montana, calls for the irrigation of 26,000 acres of Crow Indian Reservation lands by means of water diverted from the lower Yellowstone River

and the escape, should be decreased in order that the matter carried in suspension may be deposited at the head of the escape.

Escapes should be located above weak points, as embankments flumes, etc., in order that the canal may be quickly emptied in case of accident. Their channels should be of the shortest possible length, with sufficient capacity to carry off the whole body of water from both directions so that the canal below the escape may be emptied for repairs while the canal above is still in operation.

Arrangement. The greatest danger to a canal occurs during local rains when water is not being used for irrigation, leaving the canal supply full, while the discharge is augmented by the flood waters. It is essential, where a drainage inlet enters the canal, that an escape be placed opposite it for the discharge of surplus water. In order that the escape may act most effectively, the slope of its bed should be increased by at least a foot immediately below its head; besides this, the slope of the remainder of the bed should be somewhat greater than that of the canal, and it should tail into the drainage channel with a drop of a few feet.

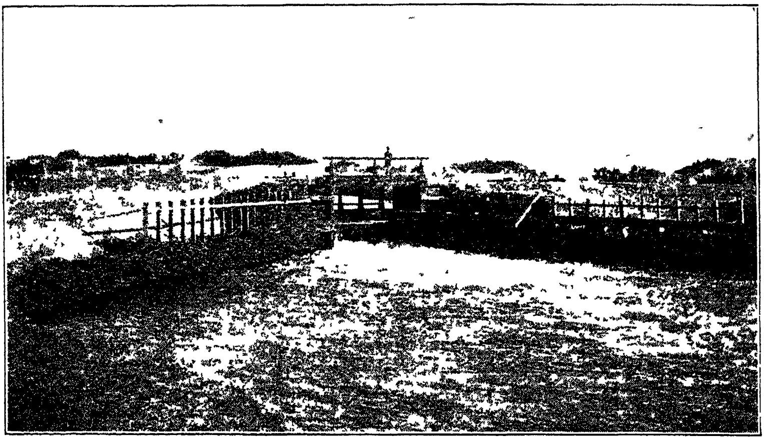


Fig 25 Drop on Main Imperial Canal at Sharps in the Imperial Valley, California, Showing Brush Wing Walls and Height of Drop

Escape heads, and the regulators necessary to their operation, are similar in design to the regulating gates at the head of the main canal. They should be as large as can be conveniently operated, and there should be a sufficient number of them to discharge the canal without unduly retarding the velocity of flow. The gates may be of timber or iron, and may be framed between piers of timber, masonry, or iron. They are operated the same as the gates at the head of the main canal; but, as the pressure upon them is not so great, the lifting apparatus need not be so elaborate.

Sand gates are practically escape gates, but they may be so designed as to be of service only for scouring and the removal of sediment. The main gates on a canal system should be designed as much for scouring purposes as for the control of the water in the canal.

Drops. At places in some canal lines it is necessary to overcome excessive erosion of the canal bed by introducing drops or chutes

Fig 26 Chute in Natural Earth, Bed Protected from Erosion by Bowlders

through which the water is carried to a lower level in a short distance. These may be constructed of timber or of masonry. Figs.

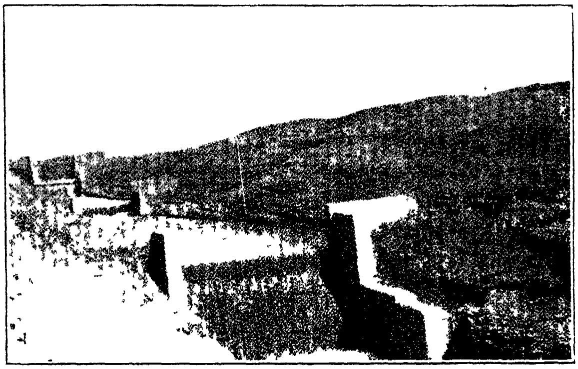


Fig. 27. Reinforced-Concrete Drops, Comanche Canal, Colorado
From Report of Colorado State Engineer

25, 26, and 27 show types of drops and chutes. Their capacity must be sufficient to allow the maximum discharge of the canal

to pass over them without raising the water upstream above the normal maximum elevation. The capacity of vertical drops can be estimated by the use of some weir formula. As in dams there should be provision made below the drop so that the falling water will not erode the stream bed. For this, a water cushion with a depth of one-third to one-half the vertical drop in the water has

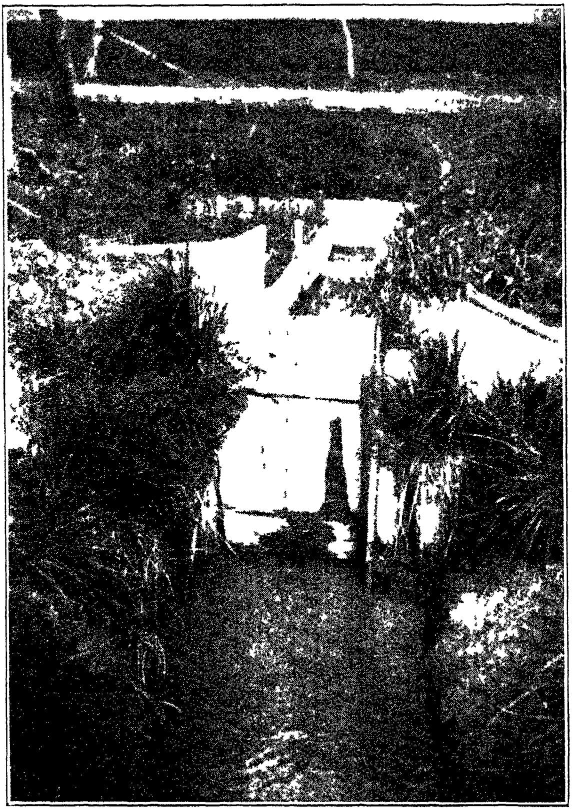


Fig 28 Typical Wyoming Turnout

been found by experience to be satisfactory. The design of a chute consists of a series of approximations, and the use of Kutter's or some other satisfactory formula.

Turnouts. The structure at the head of a lateral, used for the purpose of diverting water from the main canal, is known as a turnout. Turnouts may be open sluices for large quantities,

wooden boxes set into the banks of the canal, or closed pipes controlled by gates on the canal side. They should be placed so as to divert the required amount of water with the minimum head in the canal. Fig. 28 shows a type of wooden turnout used in Wyoming.

DRAINAGE WORKS

When the diversion line of a canal is carried around a hill or is located on sloping ground, provision must be made for side drainage; and this is sometimes one of the most difficult problems encountered in the design and construction of irrigation works. The higher the canal heads up on a stream, the more liable it is to encounter side drainage. On low ground and flat slopes, drainage may be provided for by diverting the watercourses through cuts into natural drainage areas. When this cannot be done, the drainage may be passed by one of the following methods: simple inlet dam, level crossing, flume or aqueduct, superpassage, and culvert or inverted siphon.

Inlet Dams. In case the drainage is intermittent and its volume small compared with that of the canal, the water may be admitted directly into the canal, and the excess of flow may be discharged at the first escape along the line of the canal.

Inlet dams may be built of wood, masonry, or loose stone. If the depth of the canal is small, and the height of the fall from the crest of the dam to the bed of the canal is also small, a wooden flooring may be laid in the bed of the canal, and a barrier or dam of piles and sheet piling built across the upper side. After a time, the sediment carried by the stream will fill in behind the dam to the level of its crest, and the water will then simply fall over the crest of the dam onto the wooden apron.

The inlet dam may be built as a loose-rock retaining wall, in which case the bed and banks of the canal below, as well as the opposite side, should be riprapped to protect them from erosion. If the drainage flow is considerable, more substantial works may be required, and a masonry inlet dam will be needed, a portion of the canal channel being built of masonry, and the opposite bank protected with loose stones.

Level Crossings. When the discharge of the drainage channel is large and is at the same level as the canal, it may be passed over, under, or through the latter. In the latter case the water is

admitted by an inlet dam on one side, and discharged through an escape in the opposite bank. The discharge capacity of the escape must be sufficient to pass the greatest volume of flood flow likely to enter; and a set of regulating gates should be placed in the canal below the escape, in order properly to control the amount of water passing down the canal.

Flumes and Aqueducts. The term *flume* is more generally applied to a structure intended to carry the waters of a canal around steep, rocky hillsides or across drainage lines. The term *aqueduct* is more generally applied to works of greater magnitude which are built of rather permanent material, as iron or masonry.

Construction. Flumes may be constructed of timber, steel, or reinforced concrete. In case the drainage encountered is at a lower level than the bed of the canal, it may pass under the latter, the canal passing over the drainage in the form of a flume. The drainage stream must be carefully studied as to its discharge, in order that the capacity of the waterway under the flume may be ample to pass the maximum flood discharge.

Particular attention must be paid to the stability and permanency of the foundations of the flume; and they should not unnecessarily obstruct the waterway, as this would cause a high velocity of flow in the drainage channel, which might result in scouring the bed and possibly destroying the work. In connecting the ends of the flume with the canal banks, care must be exercised that leakage does not occur at such points.

The length of a flume or aqueduct may be shortened by making the approaches at either end of earth embankments, causing the canal at the end of the flume to flow on top of an embankment, which must be of the most careful construction and of sufficient width so that it will not settle greatly or be washed away. The embankments should be faced with masonry abutments and wing walls at their junction with the flume, to protect them from erosion. The cross-section of the flume may be diminished, and in consequence its slope may be made somewhat greater than the canal at either end, so as to carry the required volume of water. This will result in a considerable saving in the cost of construction.

Bench Flumes. Bench flumes are built on steep hillsides to save the cost of canal construction. Structures of this kind should

not be built on embankments, but in excavation or upon trestles to avoid the danger of subsidence and consequent destruction. The excavated bench should be several feet wider than the flume, and the flume itself should be supported on a permanent foundation of mudsills or posts.

Fig 29. Pryor Creek Crossing on Huntley Main Canal, Huntley Project, Montana, on Line of Northern Pacific Railroad

At the point beyond the turn where the canal disappears under the crossing, it is carried by means of pressure pipe underneath Pryor Creek, which in turn is carried over the canal by reinforced-concrete structure, this arrangement being necessary on account of the flood conditions of Pryor Creek View taken from above tunnel No 3

Superpassages. When the canal is at a lower level than the drainage channel, the latter is carried over the canal by what is termed a *superpassage.* (See Fig. 29.) This is practically an aqueduct, though differing from an aqueduct in some important details.

The volumes of streams that are to be carried in superpassages are variable; at times the streams may be dry, while again their

flood discharges may be enormous. The waterway of the superpassage must be ample to carry the greatest flood flow that may occur in the stream; and especial care must be exercised in joining the superpassage to the stream bed above and below, to prevent injury by the violent action of flood waters.

Constructing superpassages of wood is not to be recommended, as the alternate wetting and drying will soon result in decay. They have occasionally been built of iron, but proper precautions must then be taken to provide against expansion and contraction, since at times the amount of water flowing in the channel will not be sufficient to maintain the temperature constant.

Siphons. The most common practice in the United States is to carry the canal under the stream by means of an inverted siphon, though at times it may be more convenient to carry the stream through the siphon and then build the canal over the siphon in the form of an aqueduct. The proper dimensions of the siphon may be computed by means of any one of the numerous formulas for the flow of water in pipes. Siphons may be of wood—built up—or they may be of wrought iron, cement, reinforced concrete, or wooden pipe.

DISTRIBUTARIES

Distributaries bear the same relation to the main canal system as the service pipes in the city waterworks system do to the city mains. From the distributaries the irrigator takes his water supply for irrigating his crops.

Best Location. These minor ditches, or *laterals*, should not be diverted from the main canal or from its upper branches, as these should have as few openings as possible, in order to reduce the liability of accident to a minimum. The water should be drawn at proper intervals from the main canal into moderate size branches so located as to command the greatest area of land and to supply the laterals and small ditches of the irrigator in the simplest and most direct manner possible.

Wherever water is plentiful and not particularly valuable, it is conducted to the lands in open ditches and laterals excavated in the earth. Where, however, water is scarce and its cost high, the losses from evaporation and percolation should be reduced to the minimum. In such a case the laterals should consist of

wooden flumes, of paved or masonry-lined earth channels, or, in extreme cases, the supply may be conducted underground in pipes and applied to the crops from these, instead of being flowed over the surface. This results in the highest possible duty and the most effective use of the water.

The most economical location in a main canal is along a ridge, whence the water may be supplied to its branches and to private channels on either side. Such a location, however, can seldom be realized. The distributaries taken from the main lines should conform to the dividing lines between water courses; and the capacity of each will then be proportioned to the duty required of it, the bounding streams limiting the area it will have to serve.

Considerations in Designing. In preparing for the design of a distributary system, a careful topographical survey should be made of the entire area to be covered by the system; and the location should be made only after a thorough study of the topographical features. Cuts and fills should be balanced as nearly as possible, and the loss of water by percolation reduced to a minimum as far as consistent with economy of design and value of water. Provision should be made for the least mileage of channels consistent with the command of the greatest area of irrigable land on either side. Attention should be paid to drainage, that it may be secured with the least possible interference with the proposed channels and distributaries.

For the complete and efficient distribution of water, the engineer should consider the design and location of the distributaries of as much importance as the design and location of the main branches. Besides the alignment, attention should be paid to the character of the soil and to the safe and permanent crossing of drainage lines, while the capacity of the channels should be proportioned to the duty required, the cross-sectional area being reduced as the quantity of water is decreased by its diversion to private channels.

The bed of the distributary should be above the level of the bed of the canal, in order to get the clearest water from the canal, and in order that it may be kept at as high a level as possible and provide surface irrigation throughout its length. Care should be taken, in level country, that the drainage lines into which the distributaries tail should have ample capacity to accommodate any flood flow that it may be necessary to discharge into them, as other-

wise the streams might become clogged and flood the surrounding country. The slope of the distributary should be as nearly as possible parallel to the surface of the land it traverses, thus commanding the maximum area and at the same time avoiding expensive and heavy construction. To realize this condition, falls may be frequently introduced, and storm-water escapes into natural channels may be provided at intervals in the course of the distributary.

In determining the dimensions of distributaries, it must be borne in mind that the greater the amount of discharge, the smaller will be the unit cost of maintenance. A single channel the width of which is equal to that of two separate channels, will have a carrying capacity more than double that of each of the smaller channels; and the cost of patrolling and maintaining the single channel generally will be one-half that of the two smaller channels. Experience has shown that irrigation can be most profitably carried on from channels having a width of 18 feet on the bottom and a depth of about 4 feet of water. The surface of the water should be maintained at from 1 to 3 feet above the surrounding area, to provide gravity irrigation and to reduce the loss by absorption.

Details of Construction. Distributary heads should be arranged much as are the heads of main canals and escapes. They consist essentially of two parts: a *regulator* or *check* below the head on the main canal, to divert the water into the distributary; and a *regulating gate* in the distributary, to admit the proper amount of water. These heads usually consist of a wooden fluming, built in the form of an apron to the bed of the distributary, with planking to protect the banks. In this flume are built the gates, consisting either of flashboards or of simple wooden lifting gates.

Distributary channels are often open channels, but when the water is conveyed in pipes, these may be of steel, of wood, or of reinforced concrete. Steel pipe should always be coated with hot asphaltum to protect it from erosion. This class of pipe will bear internal pressures of from 100 to 200 pounds per square inch. Air valves should be introduced at all high places; blow-off valves at the proper intervals; and a standpipe air chamber may be placed at the highest point in the line.

Several types of wooden pipes are in use, but for the larger conduits, wooden stavepipes are to be preferred. The pipe is built

continuously in the trench, and the staves are formed with radial edges and bound together by means of round or oval bands of steel or iron, spaced and sized according to the pressure, Fig. 30. Two types of stavepipe are employed. In one of these (the Allen patent), the outside and inside surfaces of the staves are concentric; the staves are made to break joints, and the end joints are made tight by inserting small steel plates in saw kerfs in the staves. In the other form, polygonal staves 16 feet to 20 feet long are used, which have a

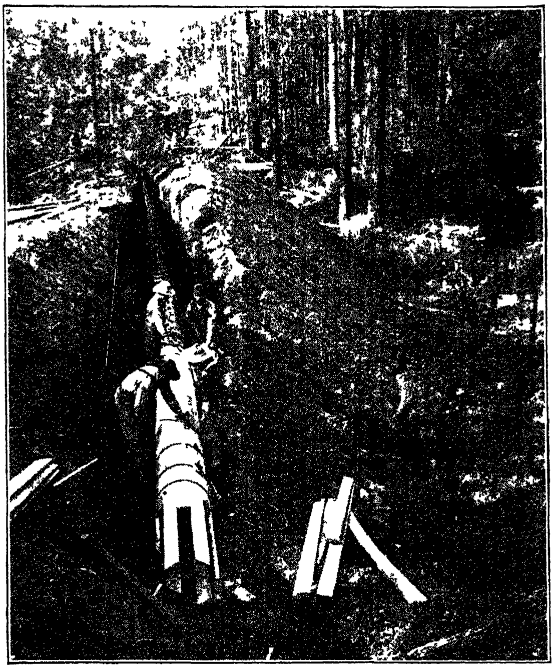


Fig. 30 Erecting a 12-Inch Pipe of California Redwood

slight tongue and groove formed on the edges. The staves do not break joints, but the end joint is made by surrounding the pipe by a layer of staves 4 feet in length.

Stavepipe is suited to pressures up to as high as 100 pounds per square inch. Above this limit it is less economical than steel, owing to the size and number of the bands required. It has been built as large as 9 feet in diameter. The staves should be made of clear stuff, somewhat seasoned. California redwood and Oregon fir are the materials most generally used for irrigation pipes in the West.

For the smaller sizes of conduits, bored pipe is better adapted. This pipe is made from solid logs, but depends for its strength upon spiral bands of flat iron wrapped tightly about it from end to end. The exterior of the pipe is coated with pitch to protect the bands and preserve the pipe. The joints are made by means of wooden thimbles fitting tightly in mortises in the ends of the pipe; and in laying, the sections are driven tightly together by means of a wooden ram. The pipe is made in sections 8 feet long and varying in diameter

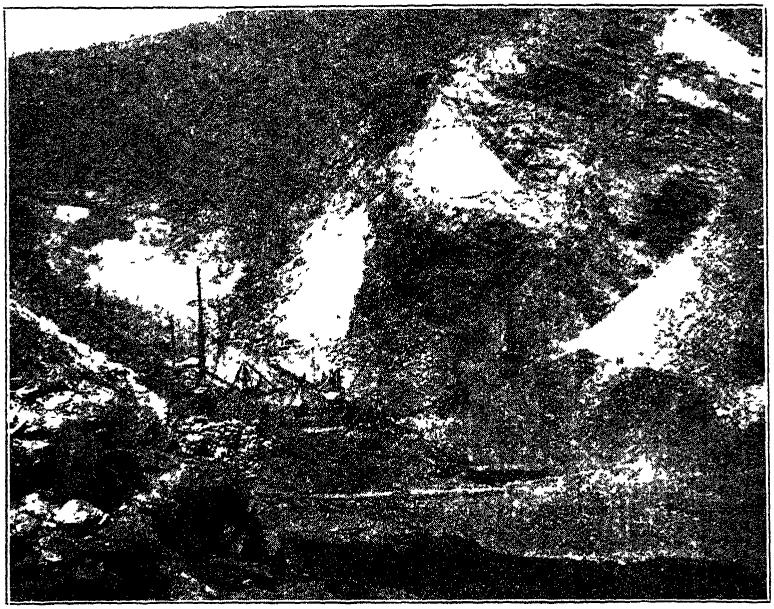


Fig 31 Site of Roosevelt Dam, Salt River, Arizona

from 2 inches to 17 inches. The bands are spaced according to the pressure; and branch connections are made by means of cast-iron specials having long sockets into which the wooden pipe is driven. This pipe is very durable, and is cheaper than cast-iron pipe, especially when transportation rates are high.

STORAGE RESERVOIRS

Classification. All varieties of natural or artificial impounding reservoirs designed for the storing up of superfluous or flood waters are termed *storage works.* Such works are intended to equalize

and insure a constant supply of water during each and every season, regardless of the amount of rainfall. They may be classified either as to the character and location of the storage basin, or as to the design of the retaining wall or dam enclosing the basin.

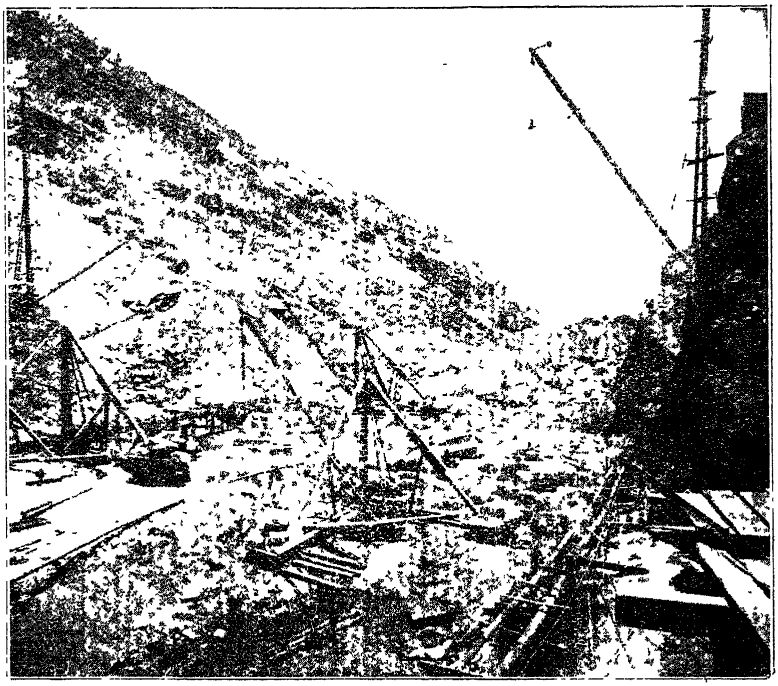


Fig. 32 Foundation Work on Roosevelt Dam, Salt River, Arizona

The Salt River project includes the Roosevelt Dam, an arched masonry gravity structure, 280 feet high and 1020 feet long, forming the Tonto Reservoir holding 1,200,000 acre-feet of water, used in reclamation of 250,000 acres of arid land near Mesa and Phoenix

Under the first classification are: (a) natural lake basins, (b) reservoir sites on natural drainage lines, as a valley or canyon through which a stream flows; (c) reservoir sites in depressions on bench lands; and (d) reservoir sites that are in part or wholly constructed by artificial methods.

Under the second classification are. (a) earth dams or embankments; (b) combined earth and loose-rock dams, (c) hydraulic-mining type of dam, or dams constructed of loose rock or loose rock and timber; (d) combined loose rock and masonry dams; and (e) masonry dams. Figs. 31 to 35 show views and plans of dams for storage works.

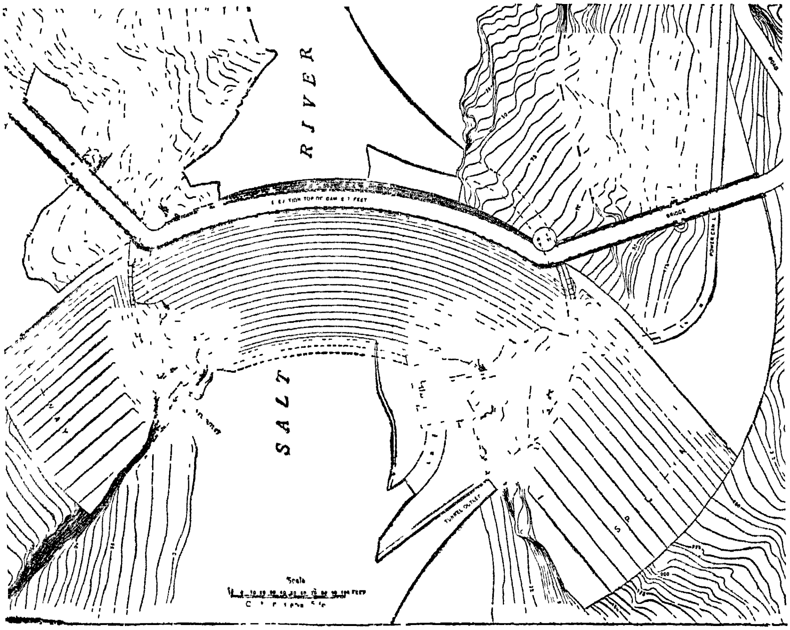


Fig. 33 Plan of Salt River Dam, Showing Location of Power House, Power Canal, Tailrace, and Contour of River Banks

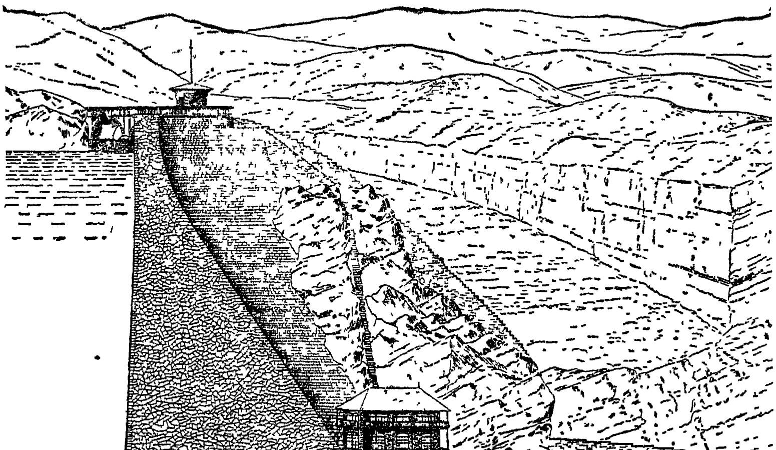


Fig. 34. Section of Salt River Dam and Elevation of Spillway and Power House

Factors in Locating. *Topography* After the position and extent of the irrigable lands have been determined, a careful preliminary survey of the surrounding region should be made in order to determine the most desirable location for the reservoir. As much information as is possible should be obtained on a catchment area, the run-off, and the maximum and minimum discharge.

The location of the reservoir site having been decided upon in a general way, a detailed survey of it should be made. This can best be done with the transit and stadia. If the valley above the dam site be long and narrow, the reservoir also will be of this shape, and in many cases two or more valleys of contributing streams may unite in one reservoir site. It is desirable that the enclosing hills be steep, and that the valley be narrow at the dam site, to avoid shallow water and expensive construction. A basin or valley with little longitudinal slope will provide a given amount of storage with less height of dam than one with a steep longitudinal slope, and for that reason is to be preferred. The larger the drainage area above the dam site, the greater the quantity of yield; and so the position of the dam with reference to the head of the area is important. The reservoir must be sufficiently high up the valley for the water to command the irrigable lands.

In conducting the surveys of the reservoir site, a tentative location of the dam should be made, and an approximate idea of its height obtained. The extreme height of the dam should be taken as the basis of the surveys, and a closed contour run out at this level around the entire site. A main traverse should be run through the lowest line of the site, from the dam to the extreme end, where it will connect with the top contour. Cross-section lines may now be run with the stadia from this line, and the topography of the site sketched by means of 5-foot contours and plotted to a large scale. Such a map will enable the engineer to determine the capacity of the reservoir for different depths of water. The dam site should be surveyed with greater care and more in detail, several possible sites being cross-sectioned and plotted to 1-foot contour intervals, to a scale of 100 feet to the inch if possible.

In deciding upon the exact location of the dam, the cost will generally be the controlling consideration. The elevation of the crest having been fixed, that location is best which requires the least

expense for excavation and construction; and this is generally where the least quantity of material is necessary in construction. With such a knowledge of the topography of a catchment basin and the reservoir and dam sites as the resulting map will give, the engineer can compute the cost of construction of dams for various heights, as well as the contents of the reservoir for these heights,

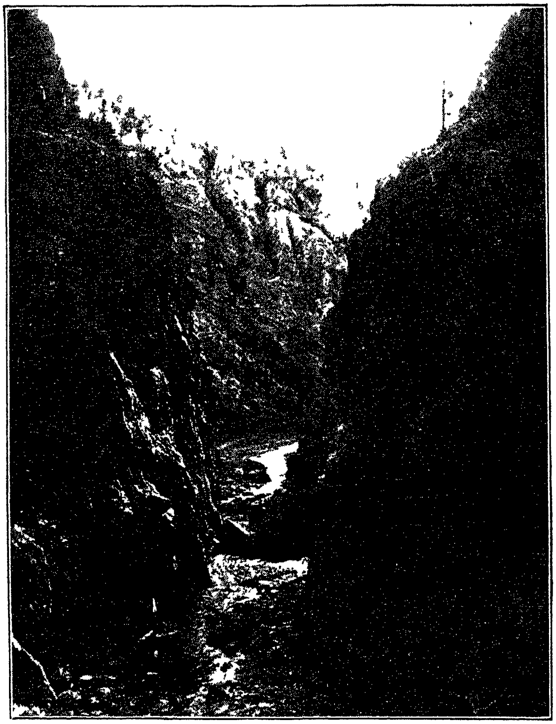


Fig. 35 Government Dam Site, Shoshone River, Wyoming, before Construction of Dam
Reservoir stores 300,000 acre-feet Height of dam 210 feet, walls of canyon 700 feet high Diamond drills bored 80 feet below river bed before striking solid foundation rock

determining thereby what height and location of dam will be the best. In many cases a straight dam at the narrowest point offers the best location, but conditions of topography are frequently met which make more economical a dam whose center line is curved upstream.

Geological Formation. Having determined the desirability of a reservoir site, both topographically and hydrographically, test

borings and trial pits should be sunk at various points in the reservoir basin, and particularly at the dam site, to ascertain the character of the soil and the character and dip of the strata underlying the proposed reservoir. (See Fig. 36.) The geological conformation may be such as to contribute to the efficiency of the reservoir, or it may be so unfavorable as to be irremediable by engineering skill. A reservoir

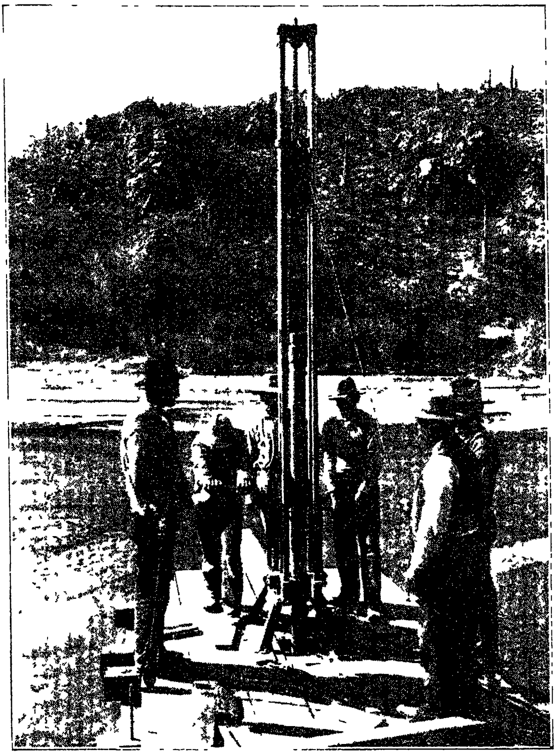


Fig. 36 Drilling in a River Bed to Find Depth of Solid Rock for Foundation of a Proposed Dam

site situated in what is called a *synclinal valley* is the most favorable, as in such a case the strata slope from the hills toward the lower lines of the valley, so that any water that falls upon the hills will find its way by percolation through the strata into the reservoir, thus adding to the storage volume. On the other hand, an *anticlinal valley* is the least favorable for a reservoir site, as the strata slope away from the reservoir site, and water falling on the hills is diverted

by percolation away from the reservoir site and to other drainage areas. A third class of geological formation intermediate between the other two, is that in which the valley has been eroded in the side of the strata that dip in one direction. In such a case the upper strata lead water from the adjoining hills into the reservoir, while the strata on the lower side tend to carry the water away from the reservoir by percolation.

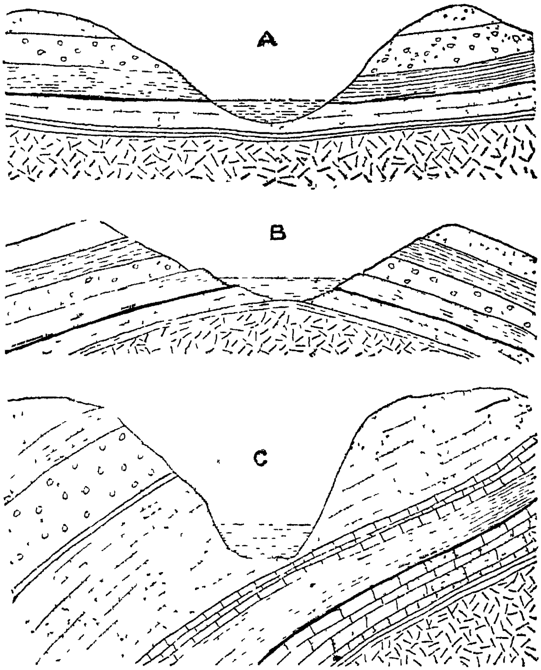


Fig 37 Types of Geological Strata for Reservoir Sites
A—Synclinal Valley, *B*—Anticlinal Valley, *C*—Intermediate Type

If the surface of the proposed reservoir site is made up of a deep bed of coarse gravel, sand, or even limestone, crevices in the last named or between the interstices of the first two named will tend to diminish the capacity of the reservoir by seepage from it. On the other hand, the geological formation may be most unfavorable, yet, if the surface of the reservoir site be covered with a deep deposit of alluvial sediment or of clay or dirty gravel, or of other equally impervious material, little danger is to be apprehended

from seepage. Fig. 37 illustrates the three types of geological strata described above.

Relation to Irrigable Lands. In considering the relation of the reservoir site to the irrigable lands, the former should be situated at a sufficient height above the latter to allow of the delivery of water to them by gravity. The area of these lands should be sufficient to require the entire amount of water stored, that the maximum return may be derived from water rates, and the reservoir should be located as near as possible to the irrigable lands, in order that the loss in transportation shall be a minimum. It may often happen, however, that the reservoir is necessarily located at a considerable distance from the irrigable lands, requiring either a long supply canal, or that the water be turned back into the natural channels, down which it flows until diverted in the neighborhood of the irrigable lands. This is very wasteful of water, since the losses by percolation and evaporation are large.

Natural and Artificial Locations. If a reservoir is situated in a natural lake basin, a short drainage cut or a comparatively cheap dam, or both, may give a large available storage capacity. The most abundant reservoirs are usually those on natural drainage lines, though these may be the most expensive in construction owing to the precautions which are necessary, in building the dam, to provide for the discharge of flood water. Almost equally abundant are those reservoir sites found in alkaline basins or depressions on bench or plain lands. The utilization of such basins as reservoir sites is comparatively inexpensive, as they can be converted into reservoirs by the construction of a deep drainage cut or of a comparatively cheap earth embankment. Scarcely any provision is necessary for the passage of flood water, and the heaviest item of expense is the supply canal for filling them from some adjacent source.

Artificial reservoirs are occasionally constructed where water is valuable, by the erection of an earth embankment above the general level of the country, or by the excavation of a reservoir basin by artificial means. Reservoirs of this kind are usually of relatively insignificant dimensions, as the expense of building large reservoirs of the kind would be prohibitive. Shallow reservoirs should never be constructed. The loss from evaporation

and percolation is proportionately great, and the growth of weeds is encouraged, where the depth is less than 7 feet, by the sunlight penetrating to the bottom.

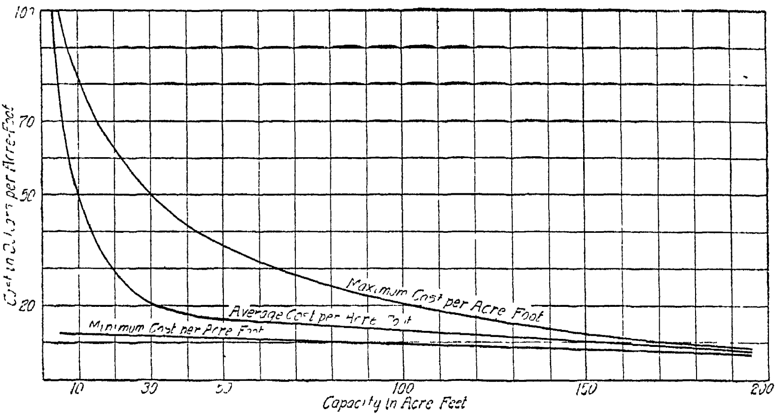


Fig 38 Diagram Showing Cost of Reservoirs

Fig. 38 shows the cost of reservoirs as given in Office of Experiment Stations' Bulletin, No. 179.

FLOW OF WATER IN OPEN CHANNELS

VELOCITY CALCULATION

Definition of Factors. If an open channel be given an inclination, the water contained therein will be set in motion by the action of gravity upon the particles. The effect of the force of gravity in producing motion will depend upon the slope, s, which is the ratio of the vertical height of fall to the horizontal distance; thus a slope of $\frac{1}{1000}$ means a fall of one foot in one thousand feet. The velocity of flow, therefore, will depend upon the length of channel l, for the vertical fall of any height h. The amount of friction offered by the sides and bed of the channel will depend upon the length and upon the nature of the material composing these, that is, upon the nature of the lining or surface of the channel in contact with which the water flows. The coefficient of velocity for a channel will also depend upon the hydraulic mean radius r, which expresses

TABLE VIII

Values of Coefficient of Roughness n in Various Channels

Character of Channel	Coefficient of Roughness n
Well-planed timber	009
Neat cement, glazed sewer pipe, or very smooth iron pipe and butt-joint wrought-iron pipe	.010
1 3 cement mortar, or smooth iron pipe	011
Unplaned timber, and ordinary cast iron	012
Smooth brickwork	013
Smooth concrete, wood troweled	014
Ordinary brickwork, or smooth masonry	015
Lap-joint wrought-iron pipe	.012 to 016
Rubble masonry	017
Firm gravel	.020
Earth canals in fair condition	.0225
Rivers free from stones and weeds	025
Canals and rivers with some stones and weeds	030
Canals and rivers in bad order	035

the ratio of the area in square feet of cross-section a to the wetted perimeter, in linear feet p. For a semicircular or circular cross-section of flow, the hydraulic radius is one-fourth the diameter.

Formulas. There are many formulas for calculating the mean velocity of flow in open channels. All of these have coefficients, and are incorrect outside of a small range of dimensions. The velocity of flow at the surface of a channel or along the wetted perimeter is generally less than the mean velocity of flow of the cross-section. The mean velocity of rivers is generally about 98 per cent of the middle-depth velocity, and from 70 to 80 per cent of the maximum surface velocity.

Chezy's Formula. The Chezy formula which is generally used as the basis of velocity formulas, is

$$v=c\sqrt{rs}$$

in which v equals the mean velocity in feet per second, c is a constant, r is the hydraulic radius, and s is the sine of the angle of slope, or rate of fall of the surface of the water.

Kutter's Formula. Values of c adapted to different conditions have been ascertained by experiment, and Chezy's formula, as

TABLE IX

Values of *c* for Earth Channels Computed by Use of Kutter's Formula

(When $n = 0225$)

Hydraulic Radius r (ft)	Slope s					
	00005	0001	0002	0004	001	01 and over
0 1	22	25	27	29	30	31
0 2	30	33	36	37	39	39
0 3	36	39	42	43	44	45
0 4	40	43	46	47	48	49
0.6	46	50	52	54	55	55
0 8	52	55	57	58	59	60
1 0	56	59	60	62	62	63
1 5	64	66	67	68	69	69
2 0	70	71	72	73	73	74
3 0	79	79	79	79	79	79
4 0	85	84	84	84	83	83
6 0	94	92	90	89	89	88
10 0	105	100	98	96	95	94
20 0	120	111	106	104	102	101

modified by Kutter, which is the one now most approved for determining the velocities of flow in open channels, is

$$v = c\sqrt{rs} = \left\{ \frac{\dfrac{1.811}{n} + 41.6 + \dfrac{.00281}{s}}{1 + \left(41.6 + \dfrac{.00281}{s}\right)\dfrac{n}{\sqrt{r}}} \right\} \sqrt{rs}$$

in which n is a coefficient of roughness for the wetted surface of the conduit, and has the values given in Table VIII.

Use of Formulas. Considerable care must be exercised in selecting the proper value of n; and an actual measurement of the flow is always to be preferred to the best formula used with the most expert judgment. Late experiments as reported in recent bulletins indicate that the values of n as given in Tables VIII, IX, and X, should be somewhat modified.

Pipes of metal or wood being of a uniform material and cross-section, the flow through them is capable of more exact determination than that through channels of earth or ordinary masonry.

Tables IX and X give values for c for a wide range of earth channels, and will cover most of the cases occurring in ordinary practice.

TABLE X

Values of c for Earth Channels
Computed by Use of Kutter's Formula
(When n = 035)

Hydraulic Radius r (ft)	Slope					
	00005	0001	0002	0004	001	01 and over
0 1	13	14	15	16	17	17
0 2	18	19	21	21	22	22
0 3	21	22	24	24	25	25
0 4	24	25	27	27	28	29
0 6	28	30	31	31	32	33
0 8	31	33	34	35	35	35
1 0	34	35	37	37	38	38
1 5	40	41	42	42	43	43
2 0	44	45	45	45	46	46
3 0	50	51	51	51	51	51
4 0	56	55	55	55	54	55
6 0	63	61	60	60	59	59
10 0	72	69	67	66	65	65
20 0	85	79	76	73	72	71

Quantity of Discharge. The quantity of discharge of a canal or river Q in second-feet is obtained by multiplying its velocity v, in feet per second, by the cross-sectional area A of the channel, in square feet, or

$$Q = Av = Ac\sqrt{rs}$$

MEASURING OR GAGING STREAM VELOCITIES

Gaging by Floats. The simplest method of gaging the velocity of a stream, when approximate results are sufficient, is by means of floats thrown into the center of the stream and timed for a given distance. These floats may be a handful of green twigs which will offer but little resistance to the wind and the velocity of the one passing over the run first taken as the maximum surface velocity. For convenience, a base of 100 feet may be measured off on the bank, parallel to the stream, and note taken of the time required by the float in passing over this distance. Several observations of the time of passage of the float over the same base may be made, or, better still, the time of passage of floats over different 100-foot lengths may be observed. The mean of these observations will

give the central or maximum surface velocity. The mean surface velocity may be determined by the use of floats in different portions of the width of the streams, and by timing the passage along a fixed length of base. The resulting velocity in feet per second, multiplied by 0.8, will give approximately the mean velocity of the entire cross section of the stream.

Measuring at Various Depths. To determine the velocity of a stream with great accuracy, the velocity should be determined not of the surface alone, but of the entire body of the stream. This may be done by timing upright rods so weighted that their bottoms float at different depths beneath the surface of the water, or by the use of current meters. The rods should be of wood or of tin, jointed so as to be used in different depths of water. Finally, the stream should be cross-sectioned at various points to determine the average area; and the quantity of discharge may then be determined.

CURRENT MEASURING METHODS

The following description of the current meter and its use in gaging streams is very complete, and is from "Irrigation Engineering", by H. M. Wilson:

> Current meters are mechanical contrivances so arranged that by lowering them into a stream, the velocity of its current can be ascertained with accuracy by a direct reading of the number of revolutions of a wheel, and a comparison of this with a table of corresponding velocities.
>
> Various forms of current meters have been designed and used, the three general classes being the *direct-recording meter*, in which the number of revolutions is indicated on a series of small gear wheels driven directly by a cog and vane wheel; the *electric meter*, in which the counting is done by a simple make-and-break circuit, the registering contrivance being placed at any desired distance from the meter, and the *acoustic meter*, in which counting is done by hearing, through an ear tube, the clicks made by the revolution of a wheel, and counting the same

Direct-Recording Meter. Of direct-acting meters, one which has been found effective in turbid waters, is the *Colorado meter*. In this, the stem is of iron pipe, several lengths of which may be joined together. This meter is difficult to handle for depths over 8 feet, or for less than 1 or 2 feet.

Electric Meter. Of the several varieties of electric meters, one which is chiefly used in the U. S. Coast and Goedetic Survey and in the U. S. Geological Survey is a modification of the *Haskell*

meter. This meter is not so good for very high velocities as that next described, as the rapidity of revolution is so great as to make counting difficult. The electric meter which has been found to work most satisfactorily under nearly all conditions of depth and velocity by the hydrographers of the U. S. Geological Survey and the U. S. Engineer Corps, is the small *Price electric current meter.* This meter undoubtedly gives the most satisfactory results in large

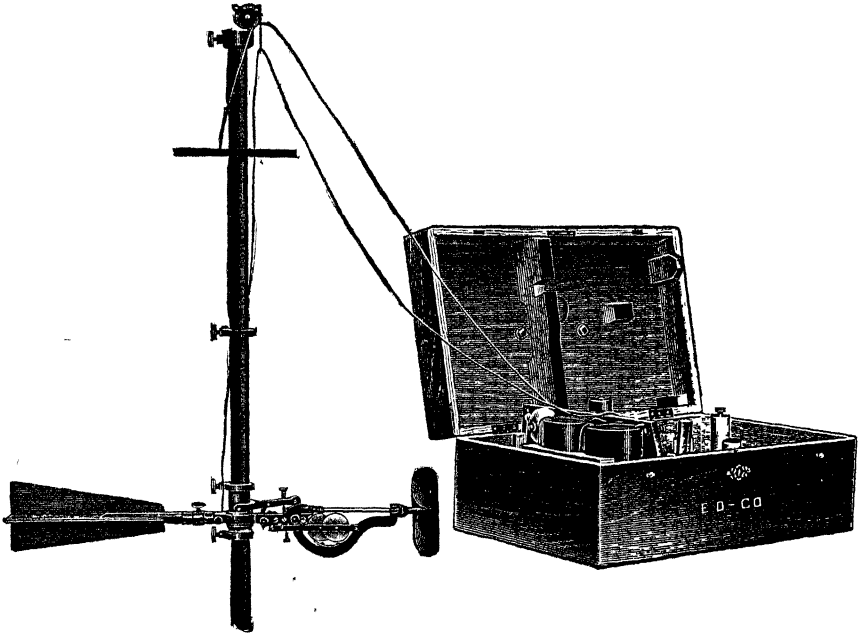


Fig 39. An Efficient Type of Current Meter
Registers mechanically in connection with electrical contact every 100 revolutions, two graduated wheels registering to 1000 revolutions
Courtesy of Eugene Dietzgen Company, Chicago

streams of high velocity. It is very accurate for streams of nearly any velocity, and is now exclusively standard with both organizations. Fig. 39 is a cut of an electric meter.

Acoustic Meter. The only acoustic meter now on the market is the *Price meter,* a modification of the electric meter, invented by Mr. W. G. Price, United States Assistant Engineer. This meter is especially desirable for its portability and the ease with which it can be handled, as it weighs but little over a pound. In very

shallow streams, it gives the most accurate results of any meter. It is held at the proper depth by a metal rod in the hands of the observer, as is the Colorado current meter.

This meter is designed especially to stand hard knocks which may be received in turbid irrigation waters, and it can be used in high velocities, as revolutions are counted by tens. It consists of a strong wheel, composed of 6 conical cups, which revolve in a horizontal plane; its bearings run in 2 cups holding air and oil in such manner as entirely to exclude water or gritty matter. Above the upper bearing is a small air chamber, into which the shaft of the wheel extends. The water cannot rise into this air chamber, and in it is a small worm gear on the shaft, turning a wheel with 20 teeth. This wheel carries a pin which at every ten revolutions of the shaft trips a small hammer which strikes against a diaphragm forming the top of the air chamber; and the sound produced by the striking hammer is transmitted by the hollow plunger rod through a connecting rubber tube to the ear of the observer by an earpiece. The plunger rod is in 2-foot lengths, and is graduated to feet and tenths of feet, thus rendering it serviceable as a sounding or gaging rod.

Use of Meter. The current meter may be conveniently used, either from a boat attached to a wire cable strung a little above a wire tagged with marks across the stream at a gaging station, or from a bridge which does not impede the channel so as to make currents or eddies in the water. Bridges, however, nearly always impede the water, and caution should be exercised when using one for a gaging station.

In using the direct-acting meter, the gager holds it in his hands by the rod, and, inserting it in the water at any desired depth, allows it to register for a certain number of seconds, or for a given number of revolutions keeping count of the time and the revolutions. This information with the use of a table will give the velocity at the given point. The meter is then held at other points and the velocities obtained. This method is known as the multiple-point method. In obtaining the mean velocity of a section at one reading, he plunges the meter slowly up and down from the bottom of the stream to its surface a few times for a given length of time, or for a given number of revolutions, at each section marked on the tagged wire,

and in this way he gets the mean velocity of each section. The area of this section must be ascertained by a cross-section made by measurement or sounding of the stream, and the mean velocity in a section multiplied by its area, gives the discharge for that section. The total discharge of the stream is the sum of all discharges through the sections. Care must be taken to hold the rod vertically, as any inclination of the meter materially affects its recording.

In using the electric meter, it is suspended and inserted in the same manner for moderately shallow streams; but in deep flood streams it is generally suspended by a wire, instead of being

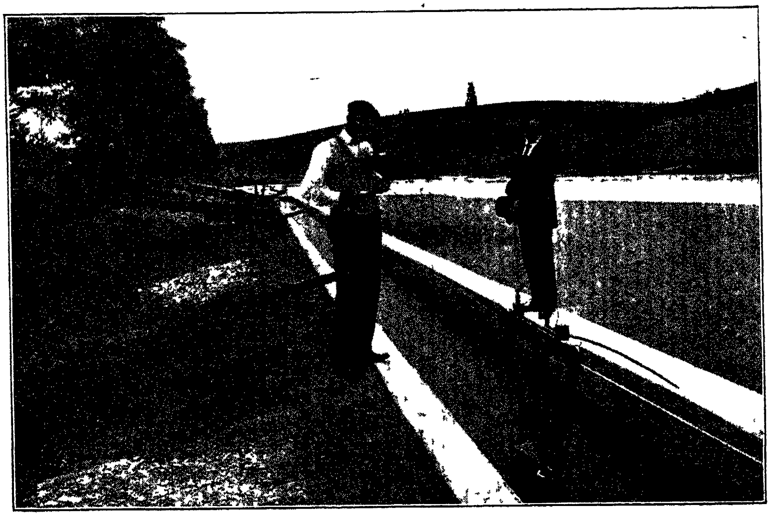


Fig 40 U S Geological Survey Rating Station at Los Angeles, California
Here the meters used in the measurement of the flow of streams are calibrated

pushed down by a rod, and a heavy weight is attached to its bottom to cause it to sink vertically.

Rating the Meter. Before the results can be obtained, each meter must be rated; that is, the relation between the number of revolutions of the wheel per unit of time and the velocity of water must be ascertained. This is usually done by drawing the meter through quiet water over a course the length of which is known, and noting the time Fig. 40 is a view of a rating station. From the observations thus made, the rating is determined either by formula or by graphic solution. The distance through which the meter is drawn, divided by the time, gives the rate of motion or velocity of the meter through

the water. The number of revolutions of the wheel, divided by the time, gives the rate of motion of the wheel. The ratio of these two is the coefficient by which the registrations are transferred into velocity of the current. This is not a constant. Taking the number of registrations per second as abscissas (represented by x), and the velocity in feet per second as ordinates (represented by y), we get the equation

$$y = ax + b$$

in which a and b are constants for the given instrument.

In determining the rating of the meter graphically, the values of x and y obtained directly from the instrument are plotted as

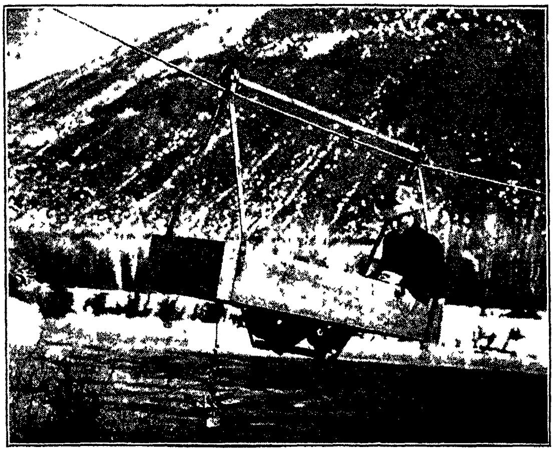


Fig. 41 Gaging the Velocity of the Stream by Means of the Current Meter

co-ordinates, using the revolutions per second as abscissas and the velocities as ordinates. In this way a series of points is obtained through which a connecting line is drawn, giving the average value of the observations. From the position of the line thus plotted the velocity can be read off corresponding to any number of revolutions per second.

Gaging Stations. The first operation in making a careful gaging of velocity by means of a current meter, is the choosing of a good station. This consists in finding some point on the course of the stream where its bed and banks are nearly permanent, the current of moderate velocity, and the cross-sections uniform for over 200 feet above and below the gaging station. At this point

a wire should be stretched across the stream and tagged with marks placed every 5, 10, or 20 feet apart, or closer, according to the width of the stream. An inclined gage rod is firmly set in the stream at some point where it can be easily reached for reading. It should be not less than 4 by 4 inches, and marked for feet and tenths of vertical depth. The gage heights with the corresponding discharges are recorded through a long period of time, in order that the variation in the velocity and discharge may be had for different flood heights. Fig. 41 shows a gaging station in a river with a cable suspending a carriage for the observer and Fig. 42 shows a temporary gaging station in a canal.

Fig 42 Measuring Discharge of Irrigation Canal with Current Meter

Fluctuations in the height of streams can be measured with even greater accuracy, than is obtainable by the readings of a gage rod as above described, by using a *Nilometer*, which is a self-reading gage. The chief objection to the use of the instrument is that its maintenance requires the attention of a person of considerable mechanical skill, in order that it may be kept in proper order. There are three general forms of Nilometer employed by the hydrographers of the U. S. Geological Survey. These have horizontal recording cylinders, vertical recording cylinders, and vertical record discs. All of these devices are driven by clockwork, and are designed to run a week or more before removal of

the recording paper. The record of stream height by the Nilometer is on a scale less than the actual range of the water, and the recording pencil is connected by a suitable reducing device with a float which rises or falls with the stream. This float is usually placed in a small well near the stream bank, its bottom communicating with the stream bed by a pipe of such size as will not readily become clogged. The fluctuations of the water in this pipe correspond with those of the stream, and turn the recording wheel through the agency of a cord wound around the wheel and having its lower end attached to the float.

Rating the Station. After daily readings of the gage height of the water have been taken at the station for some time, and the velocity measured by means of the meter at different heights of stream, the results should be plotted on cross-section paper, with the gage heights as ordinates and the discharges (obtained by multiplying the velocities into the cross-sections) as abscissas. These points generally lie in such a position that a curve drawn through them somewhat resembles half of a parabolic curve, and represents the discharge for different heights. Having once plotted this curve, it becomes possible to determine the discharge of the stream at any time by knowing the height of the water on the gage rod.

Measuring Weirs. The simplest method of measuring the discharge from canals and streams of moderate size, is by means of weirs. It is extensively used in Colorado and other portions of the West, and especially commends itself to the use of irrigators, because of the simplicity and cheapness of construction of the weir, its accuracy of measurement, and its ease of operation. The results of weir measurement are easily interpreted by use of tables giving quantities of flow directly in second-feet.

By a *standard weir* is meant one in which the inner face is a vertical plane, and the edge of the weir, or its crest, is sharp. Weirs are generally made rectangular; and the crest of such a weir may extend to the side of the flume or canal conducting the water to it, or it may be a rectangular notch cut in the weir plate, the vertical edges being beveled similarly to the horizontal edge. In the former case, the contractions are said to be *suppressed;* in the latter, the weir is said to have *end contractions.* If the contractions are not suppressed, the weir plate should extend in a plane on each

side of the weir a distance at least 3 times the depth of water on the crest, and about an equal distance below the crest whether there be contraction or not. The depth of water on the crest is measured on the surface of the water before it begins to curve downward toward the weir, at a distance up the stream or out to the side on the bulkhead equal to three or four times the head on the crest. Fig. 43 is a cut of a rectangular weir.

Francis Formula. As deduced in hydraulics the theoretical formula for the discharge of a weir is

$$Q = \frac{2}{3} m \sqrt{2g}\ l h^{\frac{3}{2}} = c l h^{\frac{3}{2}}$$

In the experiments conducted by Mr. J. B. Francis, a value for the constant c was found to be 3.33 for weirs without end con-

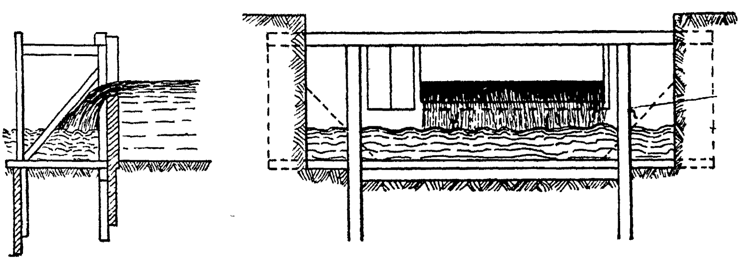


Fig 43 Rectangular Measuring Weir

tractions, which gives the following formula for the discharge over a weir without end contractions:

$$Q = 3.33\ l h^{\frac{3}{2}}$$

For weirs with end contractions the formula is given as

$$Q = 3.33\ (l - 0.1\ nh)\ h^{\frac{3}{2}}$$

which is known as the *Francis Formula.* In this formula Q is the discharge; l is the length of the crest of the weir; n is the number of end contractions; and h is the head on the crest, all units being in feet and seconds. The constants in this formula were obtained by the use of long weirs and heads of less than one-third the crest length. In the absence of a more accurate formula, this one has been used to estimate the discharge of small weirs, such as are usually

found in use for measuring irrigation water. Recent experiments conducted by the U. S. Department of Agriculture indicate that the values obtained by the Francis formula for small weirs do not agree so very closely with actual discharges, but it can be used for approximate estimates in the absence of a more exact formula or an exact table.

In using the ordinary weir, it should be placed at right angles to the direction of flow of the stream, the upstream face being in a vertical plane. The crest and sides should be chamfered so as to slope downward on the lower side with an angle of not less than 30 degrees, while the crest itself should be horizontal and the ends vertical. The dimensions of the notch should be sufficient to carry the entire stream and still leave the depth of water on the crest not less than 7 inches.

In the application of the formulas to determining the quantity of flow over a weir, attention should be paid to the following details:

The water shall not be more than 24 inches, nor less than 7 inches, in depth, the depth of water on the crest shall not exceed one-third the length of the weir; there shall be complete contraction and free discharge, the water approaching without perceptible velocity and free from cross currents. The distance from the side walls to the weir opening should be at least three times the depth of water on the weir; and the height of the crest above the bottom of the channel should be at least twice the depth of the water flowing over the crest. Air should have free access under the falling water, and the channel of approach should be straight and of uniform cross-section.

For some streams it is impossible to comply with all the conditions enumerated and only approximate values can be obtained by the use of the formulas given. More exact data, as yet unpublished, will probably be available in the near future.

To determine the depth of water flowing over a weir, a post should be set in the stream a short distance above it, and to this a gage rod, suitably marked, should be attached. For very accurate measurements a hook gage should be employed, consisting of a hook attached to a divided rod and fitted with a slow-motion screw and vernier. The hook is below the surface of the water; and by turning the slow-motion screw it may be raised until the point of the hook is just in contact with the surface, as indicated by a slight elevation of the surface at the point of the hook. The difference of elevation between the hook and the weir may be taken with a leveling instrument.

MEASUREMENT OF CANAL WATER

In order that water flowing in open channels may be sold by quantity, it is necessary that the volume admitted to the canal shall be readily ascertained at any time, and that the method of admission shall be so regulated that it cannot be tampered with. No method has yet been devised for accomplishing this easily and cheaply; and as a result, water is almost universally disposed of by canal owners by means other than direct sale by quantity. In this country, rentals are charged per acre irrigated, rather than by the amount of water required for the irrigation.

Essentials for a Module. Prof. L. G. Carpenter specifies the following conditions as most desirable for *a module*, or apparatus for measuring water for purposes of irrigation:

Its discharge should be capable of conversion into the common measure, which is cubic feet per second. The ratio of discharge indicated from two outlets should be the actual ratio The same module should give the same discharge wherever placed; it should be capable of being used on canals of all sizes, and of being set to discharge any fraction of its capacity for the process of distributing pro rata. Attempts to tamper with or alter its discharge should leave traces easy to recognize, and it should be simple enough to be operated by men of ordinary intelligence, so that calculations should not be required to regulate the discharge of different modules or to determine the amount thereof. It should occupy but small space, and the discharge should not be affected by variations of the water level in the supplying canal. It should be inexpensive, and cause the least possible loss of head. Nearly all modules attempt to maintain a constant pressure of water above the opening, the orifice remaining unchanged.

Factors of Measurement. The following upon the measurement of water is from "Irrigation Institutions", by Elwood Mead:

The distribution of water for irrigation is attended by many perplexing conditions. Streams vary in volume from day to day. Wells which cannot be lowered in April, often fail in August. The water supply is subject to continued waste and loss. It sinks through the bottom of the canal by seepage, and is taken up by the air through evaporation.

When the supply was abundant and the seepage limited, these vicissitudes were of small importance; but with the growing use of water, changes in methods and policies are necessary. This is especially true regarding the care taken in its measurement, and in the attention now being paid to the contracts under which it is supplied to irrigators. When streams carried more than was needed, water was seldom measured. Canal companies took what they wanted, and the irrigator was charged for the acres irrigated without any reference to how much he used. The result of this lavishness does not warrant its continuance. It led farmers to substitute water for cultivation, and to injure their land and exhaust streams by wasteful and careless methods.

Units. The need of a definite unit of measurement for the commodity bought and sold is now manifest. Without this there can be no satisfactory basis for transactions in water, or any intelligent or certain measure of value for irrigation purposes In the establishment of such a unit, several things have to be taken into account. It should be in accordance with the requirements of agriculture, so that the quantities to be measured can be regulated by simple and not too costly devices, and be stated in a unit convenient of computation Any unit, to be generally adopted and enforced, has to be both feasible in operation and in accord with the needs or prejudices of water users. Water cannot be delivered to irrigators by the pound or ton Three units of measurement are now in general use, and some one of these three is recognized in the laws of nearly every arid state, and is nearly always stipulated in water contracts They are the *inch*, the *cubic foot per second*, and the *acre-foot*.

In the measurement of water for irrigation, there are two distinct principles involved which it is desirable to have clearly defined and to keep separate in the mind The first is the unit of volume to be employed, wholly apart from the method by which this unit may be measured in actual practice. Thus, in irrigation, if we say that the unit of measure is the cubic foot per second, the character and volume of the unit are not affected whether water is measured by the flow over a weir, or through a flume, or by the strokes of a pumping engine The unit may sometimes be the quantity of water which issues from an opening of fixed dimensions, with or without pressure; or the unit may be the acres of land irrigated, under certain conditions

Measuring by the Inch. In some cases, however, the unit of measurement is associated with a special device or instrument by which it is to be actually determined. The form of this apparatus should be in accord with the principles of Hydraulics, and be determined by scientific considerations The inch is such a unit of measurement; it has to be associated with some particular device or instrument of measurement Its use is as old as irrigation. In this country it is older than modern irrigation, having been first used by the placer miner, and borrowed from him by the irrigator In both mining and irrigation, it is the volume of water which will flow through an inch-square orifice under a uniform and designated pressure. The slope and size of the orifice and the pressure upon it are fixed by law in a number of states, and in others regulated by custom.

The ruling custom in the United States is to have the orifice, through which water is delivered, 6 inches in height and wide enough to deliver the required number of inches The pressure on this orifice varies from 4 inches above the center in some places, to 6 inches above the top in others In Nevada, the inch has sometimes an opening 4 inches in height, with a pressure of 6 inches above the top. Irrigators who are not able to compute the quantity of water flowing over weirs or through flumes, prefer, as a rule, to have their water measured by the inch. They can tell by looking—or believe they can—whether or not the quantity contracted for is being delivered; and when the conditions presented by the statute are complied with, they can tell, with a close approximation to the truth, whether or not they get what they pay for.

The most serious objection to this unit is the name. Men accustomed to square inches and cubic inches, confuse them with miner's inches and statute inches. Because of the confusion, they frequently determine the inches of

water being furnished them, by ascertaining the number of square inches in the cross-section of their ditch or lateral, and calling this the number of inches of water received, although in doing so they disregard both the absence of an orifice, the pressure upon it, and the grade or velocity of the stream measured It is the common practice on many streams in Utah, for the water masters to measure the inches of water in the ditches by taking the cross-section of their flow, and wholly disregarding pressure and velocity

A simple device for measuring miner's inches consists of a board 2 inches thick, 12 inches wide, and about 8 feet long The opening is 1 inch wide and 50 inches long, and the distance from the top of the board to the center of the opening is exactly 4 inches on the upstream side On the downstream side, the opening is beveled so that the whole presents sharp edges to the stream A sliding board is hung upon the top of the first board, with a strip screwed along its upper edge, this sliding board being wide enough to cover the opening on the upstream side. In the slot, there is a closely fitting block made to slide on the beveled edges, and fastened by a screw to the sliding board When the sliding board is moved backward or forward by means of its end, which is extended for a handle, the block moves in the slot and determines the length of the opening

When used to determine the flow of a stream, the board is placed so as to dam the flow completely, and the sliding board is moved backward or forward until the water is all passing through the slot, the water being kept to the top of the board, or 4 inches above the center of the opening The length of the opening measures the number of miner's inches of water flowing through If the flow is too great to pass through the opening 1 inch wide, the opening may be made wider, the water still to be kept 4 inches above the center of the opening.

Many measuring boxes in European canals are constructed in the most substantial manner, of masonry. The orifice is cut through stone, with edges of metal, and with the utmost precision in its dimensions. Thus far, in this country, but little attention has been paid to accuracy, either in the form or size of openings, although much ingenuity has been shown in designing automatic regulators The pervading practice in the west is to make the measuring boxes of wood, and to give slight regard either to the freedom of delivery or to securing uniform pressure One of the reasons why no more consideration has been given to the accuracy of measuring devices is the fact that the conditions of water contracts are so often not in accord with the way water has to be used

The field of usefulness of the inch is restricted to the measurement of comparatively small quantities of water It is well adapted to the distribution of water to irrigators, from canals or from the main laterals of canals, but it is not suited to the measurement of rivers or to the distribution of water from a river. Where large volumes, or widely fluctuating volumes, are to be measured, the construction of a satisfactory device for measuring by inches is not practical. There are a number of canals in this country which carry from 50,000 to 125,000 statutory inches It is manifest that while the width of an orifice can be extended indefinitely without materially affecting the accuracy of the measurements, every change in its depths must materially increase the velocity, and hence the quantity of water discharged by each square inch of its cross-section Nearly all of the statutes prescribe a maximum depth for the orifice, and require that increase in volume delivered shall be secured by extending its

length. To measure the water required to fill the Del Norte Canal, would require an opening 1736 feet in length, which would be practically impossible.

The limitations of mechanical devices render the inch unsuited to measuring the flow of rivers. In states where the inch is recognized as the legal unit in the distribution of water among irrigators, some other has to be employed in the measurement of the flow of streams

Cubic Foot per Second. The cubic foot per second has come into general use as the unit of volume for gaging and dividing weirs, and in measuring the flow of ditches and canals Nearly all of the arid states and territories have made it the legal unit in water contracts, and for defining the amounts of appropriation from streams It has the double advantage of precision in statement, of being well adapted to the measurement of large as well as small volumes of flowing water, and of permitting the employment of varied methods of measurement. In many states it is used in connection with the inch. The flow of the stream and the amounts of appropriations are stated in cubic feet per second. The water, after it is turned into the ditches, is measured out to farmers in inches. This renders it desirable that there should be some basis of comparison, some legally defined ratio between the inch and the cubic foot per second

A number of states have passed laws fixing the number of inches which equal a cubic foot per second Legislation fixing the ratio has been of decided service in the states where the inch is still employed

The following is the ratio assumed by law or custom in a number of states:

Equivalents of 1 Cubic Foot per Second

	(in)	
Colorado	38 4	(statutory)
Montana	40	(statutory)
Idaho	50	(miner's)
Arizona	40	(miner's)
Nevada	40	(miner's)
Utah	50	(miner's)

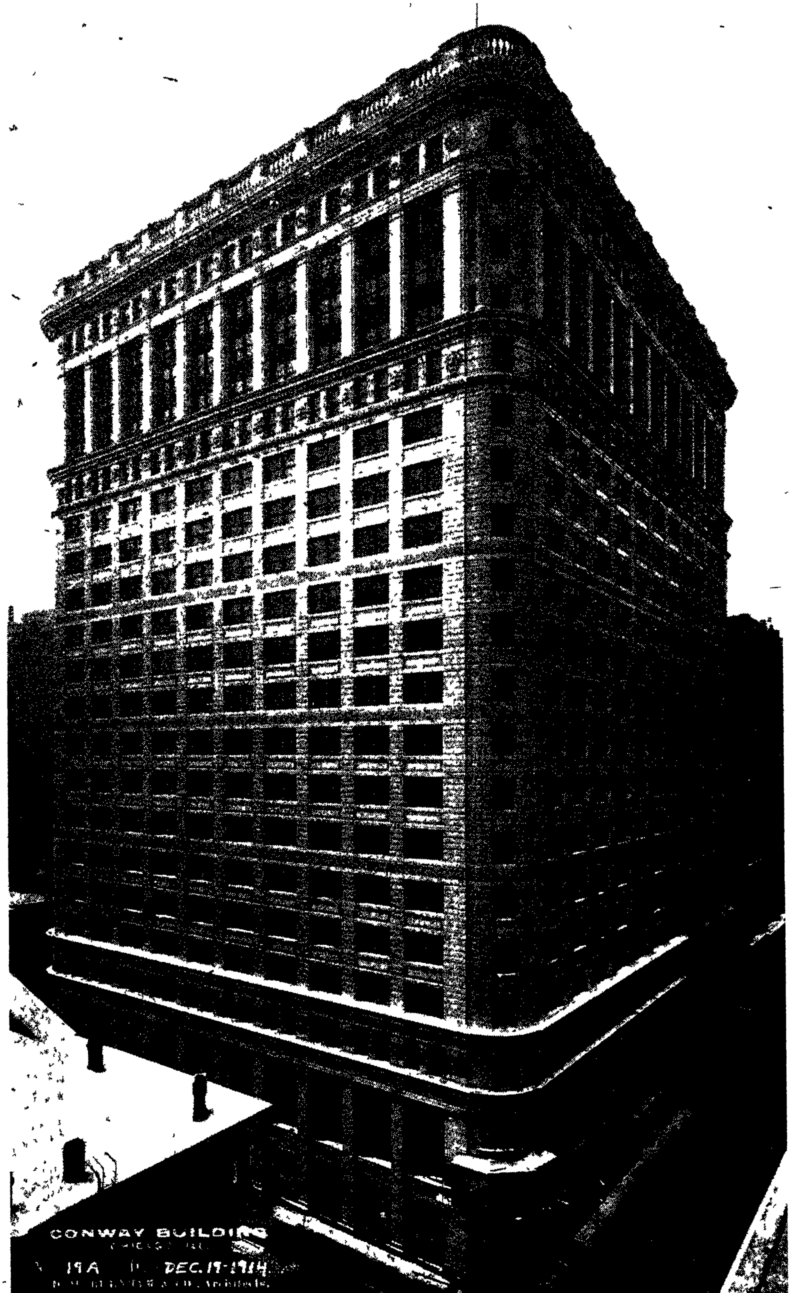
In many places the inch is retained as a term where it has no existence in fact The farmers who have been accustomed to estimating the flow of water in inches find it hard to think of this flow in cubic feet per second Because of this, engineers who measure the flow of ditches or canals in cubic feet per second convert this into inches according to the statutory ratio. On many ditches where the inch is still retained as the unit of measurement, there is no measuring box for its delivery. Farmers pay for their water supply in inches, and estimate the flow in their laterals. Where the water is measured, the volume is determined in cubic feet per second, and converted into inches on some arbitrary ratio. The real unit is the cubic foot per second.

Irrigating Stream. The irrigating stream is a unit in common use in Utah It is a stream which one man can control to advantage, but no rules for its measurement have ever been prescribed. The size of the stream is left to the water masters, who are charged with distributing water to the farmers. The following extract from a notice sent out by a Utah canal company illustrates the use of this unit:

"The books of the company show that you are the owner of ------shares of stock, and you will therefore be entitled to the use of an irrigating stream for------hours."

The water in this canal is not measured, nor are the diversions. The water master estimates the number of streams in his canal, and these streams are used in turn by the farmers. They are supposed to be equal, but measurements made upon a canal to determine the accuracy of the judgment of the water master, gave widely varying results.

The amount of money paid for water each year by irrigators is so large that it seems surprising that they have not paid more attention to the accuracy with which it is measured. In transactions involving any other kind of property, care is taken to see that it is accurately measured, but, although water costs more than any other commodity used by the irrigator, it is bought and paid for without either buyer or seller knowing how much is delivered. The need of greater accuracy in water measurements has led to the passage of a law in Utah requiring the State Engineer to give information and advice about the placing of measuring devices. The State Engineers of Colorado and Wyoming are required to advise irrigators in respect to the measurement of water, and a recent Colorado statute also provides for the use of registers which will keep a continuous record of the quantity of water delivered. The introduction of registers and greater accuracy in the construction of measuring boxes, is one of the developments of the near future. Their installation will do much to reform water contracts, prevent the awarding of excessive amounts of water in decrees, promote economy and efficiency in use, and extend the reclaimed area.



CONWAY BUILDING, CHICAGO
D. H. Burnham and Company, Architects
Courtesy of Thompson-Starrett Company, New York City

COST-ANALYSIS ENGINEERING

PART I

WHAT IS COST ANALYSIS?

INTRODUCTION

"For which of you, intending to build a tower, sitteth not down first and counteth the cost, whether he have sufficient to finish it" (Luke, 14 28)

Cost Analysis Defined. Cost analysis is proved by this quotation to be not a new thing. According to the Century Dictionary, which defines "cost" quite to the writer's mind, "*Nothing has any cost until it is actually attained or obtained; while price is the amount which is asked for a service or thing*" Therefore, we estimate the cost, in doing which we must analyze the particular proposition we have before us.

Good Judgment a Necessary Element. By the very meaning of the term, "cost analysis" is, therefore, in no sense a new idea, being merely a modern designation for good judgment based upon experience, or the analysis within the brain of the individual as to the relative costs of different classes of work, a judgment and decision which are reached through an intimate acquaintance with costs and the value of work along various lines of construction. By consulting the chart, Fig. 1, a good idea may be obtained of how intimately cost analysis is connected with the other branches of an engineering contract business and to what extent the cost analyst draws upon these departments for material to help him form his good judgment.

Since the days when things were first built, the individual workman has accumulated a mind full of ideas of relative costs of work, based upon his own impressions and experiences. Those in whose brain this judgment has received its highest development are the ones who have advanced from the position of laborers to foremen, to superintendents, and finally to the position of employers.

Many a man without education has become a successful contractor or manufacturer because, by years of hard labor and experience he has accumulated a knowledge of costs that has enabled him to make propositions which are above criticism as examples of con-

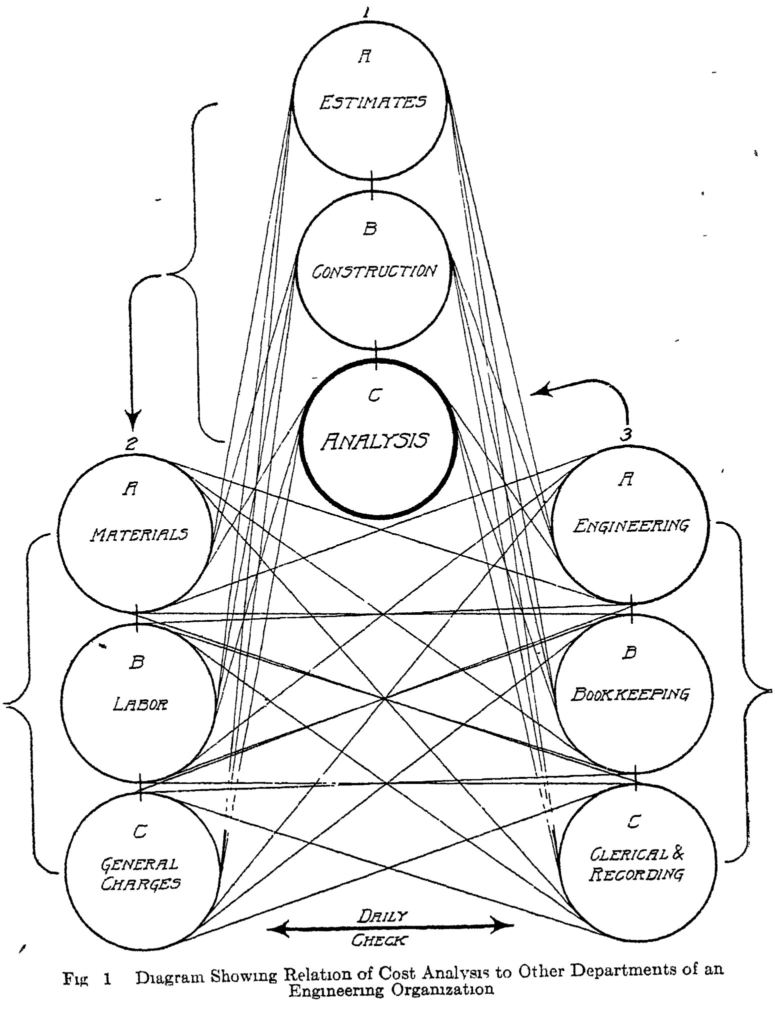


Fig 1 Diagram Showing Relation of Cost Analysis to Other Departments of an Engineering Organization

servative bidding and as evidence of the real cost of the proposition, and which are really nearer the truth than the estimates prepared on the basis of data collected from preceding lettings and theoretical costs.

Too strong emphasis cannot be laid upon this particular point, for herein lies absolutely all the value of cost analysis. The mere cost of a proposition when reduced to dollars and cents is of little value, except when used with the good judgment attendant upon actual experience and a personal acquaintance with the nature of the work.

Experience the Teacher. The cost-analysis turn of mind is not something to be acquired, unless the individual has within himself an instinctive disposition to ascertain why the results are as recorded. The laborer, no matter how ignorant and uneducated, who advances to the position of a business of his own, is a man who grasps this fundamental point, and he, perhaps unconsciously, gains the best idea of the value of work in relation to the circumstances surrounding it. In other words, such a man has the advantage of not being encumbered with reports which influence his judgment, but of forming his conclusions entirely with regard to the circumstances and conditions under which he must work.

Too often cost records compiled according to the modern idea as a basis for future operations, obscure these points, costs being used which are in no sense indicative of the actual work. Under the method mentioned, in which the laborer, by his own efforts, advances to the position of employer, and in so doing grounds his knowledge absolutely on experience, progress for the successful man is necessarily slow. The individual who begins with insufficient experience is at a great disadvantage, and unless he meets with very unusual or fortunate circumstances the result may be discouraging, or even a failure.

Natural Growth Inadequate for Present Day. In the new or undeveloped country, or in a country where the development is slow, such a natural growth is the proper one, and one that is quite sufficient. The idea of the apprentice, the journeyman, the master workman, and the employer is one which entirely satisfied the conditions of the Middle Ages, but is utterly inadequate to meet the demands of our modern everyday life. In the old days, the artisan in any special line had accumulated within his brain all the details necessary successfully to complete any work. In our day blue prints are supplied to the contractor, and if he cannot read a plan or estimate quantities from a blue print, he is at a serious disadvantage. Modern cost analysis demands, therefore, not only

a record of actual results, but the ability to read and interpret a plan for the purpose of applying such practical information to the problem in hand.

Standardizing of Methods Important. In individual work minor changes may be possible and individual ideas may be incorporated, but, in general, any uniform system of cost analysis can be obtained only by the working-out of a standard method of cost distribution, so that when one job is analyzed and compared with another, or when the individual making the analysis is not familiar with either job or at least with only one, there may be a direct method of comparison and analysis.

RELATION OF COST KEEPING TO COST ANALYSIS

Overlapping of the Two Subjects. Cost analysis is so closely identified and connected with the subject of cost keeping that, to a certain extent, the reports will probably overlap at many points. Any broad comprehensive system of cost keeping will contain certain elements of cost analysis; on the other hand, cost analysis, being based upon cost keeping, must necessarily embody suggestions along the line of proper cost keeping.

The distinction between cost keeping and cost analysis should be thoroughly understood, and to make this distinction more clear the following definitions are given:

(1) *Cost analysis is the executive mapping-out of new work, supervision of such work, and compilation of the records obtained through cost keeping in order to secure comparative values.*

(2) *Cost analysis is the study of the data obtained by cost keeping whereby the reasons for the results are determined.*

(3) *Cost analysis is the executive department of contract work for the purpose of properly supervising and securing intelligent records, whereby experience is fortified for future reference.*

General Remarks. Cost keeping must involve the preparation of forms or blanks by which distribution of the pay roll, material, etc., is properly made to the various accounts; but as this work is generally in the hands of only ordinary clerical help, and in the majority of cases is done under adverse conditions, no particularly intelligent idea is to be expected from such records unless the executive head maps out the work of separating the items to a degree far

beyond mere forms. For example, concrete work is "concrete work" to the cost-keeping man, regardless of its character, how it is done, where it is done, the time of the year, or a hundred other conditions which may make the reports misleading. Many maintain that by adding a complete description of the work, this difficulty may be overcome. This is true to a certain extent, but if the description is made long enough to be of any particular value, the cost record is cumbersome and of little use to the average busy man. There is connected with every job—no matter how done or where done, even if under abnormal conditions—a method of so dividing and subdividing the items that the same items of work on jobs of entirely different character may accurately be compared, provided, of course, that a standard method of distribution is used.

Cost analysis is not concerned with what work cost, but is concerned with why it cost what it did, and whether this cost is above or below the average, and if so, why? Cost keeping states certain absolute facts as such because these facts are records. Cost analysis goes one step further and states why the work cost a certain sum and what reasonably could be expected under different circumstances. Cost analysis begins at the time estimates on new work are prepared, superintends the adoption of a cost-keeping system in harmony with the estimates, watches the work during its progress for all abnormal conditions and segregates them, and finally analyzes the records.

In successfully carrying out the idea of cost analysis, cost keeping itself is affected by the character of the records required. In order to avoid confusion in the cost records all "specials" are recognized and so recorded with reference to any particular job, thus separating them from the standard elements which enter into all work. This means that before operations are actually begun, estimates and plans must be studied and special distribution provided for any class of work which is peculiar to this job; furthermore, if during the progress of the work, any special peculiarities arise, these must be properly recognized in the records.

Simple Methods. Cost analysis should simplify, rather than complicate, cost keeping. Cost keeping may be carried to an extreme in any one general division of the work, being so subdivided as to make it almost impossible to keep the work up day by day with

the ordinary clerical force. Such an elaboration of divisions produces misleading distributions and records. Cost analysis does not contemplate any such method; in fact, it does not contemplate to any extent the subdivision of the general items of work except under special circumstances. If the specials and other peculiarities of any job are recognized and carried under special headings, then all the remainder may be carried as standard divisions of work.

To illustrate—on a job of concrete we are not interested, except in special cases, in knowing what it costs to drive nails and draw nails, or to saw off boards. To carry the point a step further—we are not interested in knowing what it costs to wreck forms after the work is done. If we have sufficient information as to what it cost to make the forms the first time, then, knowing the cost of the form work for the entire job, and the number of times the forms were used, we have all the information needed for future estimates and comparisons, providing that, if there is any special form of construction, that has its own unit distribution.

General Conditions Affecting Cost Keeping. The subject of properly recording cost is a very difficult and complicated one when applied to construction work; it is not only complicated, but in some cases it is exasperating and seems almost impossible of application.

Regarding Shop Labor. The subject of cost recording is not so difficult in its application to manufacturing plants, where we have an industry within the limits of the four walls of a building, and where the men are always in sight and subject to observation and to a close check upon their output. The mechanic producing any part of a machine on a lathe or a planer, is probably doing the same class of work, day in and day out, and can be watched very closely; the record is, therefore, largely a question of counting the number of pieces produced and dividing the cost, to ascertain the cost per unit.

If, in addition, the plant is a large manufacturing concern where more than one man is employed on the same class of work or on the same class of machines, the output of the different men can be ascertained, the less productive operatives weeded out, and the general average output determined. Labor working under these conditions is largely independent of the weather, is generally of a

high grade, and the facilities for a close check upon the actual time employed are correspondingly increased.

Regarding Construction Labor. In the case of construction work, however, a large percentage of the payroll will be made up of common labor, in many cases of foreigners to whom it is impossible even to explain what is desired. All labor is exposed to the inclemencies of the weather, and the output of the men is thereby directly affected.

Furthermore, the circumstances surrounding the work, and the very nature of the work itself, make it difficult to sift out the bad labor from the good, except through the medium of the foreman, and the output is measured more from the gang unit than from the individual unit. As a result, the laborer is not hired for any particular line of work, except in special cases, but is shoved around from one class of work to another, so that at the best all that can be expected is an approximate estimate of the distribution of his time, and the work accomplished. Many times he may be placed on work where there is absolutely no unit measurement of output.

The character of the work handled, the class of labor, the numerous large gangs with frequent discharges and additions of men, serve, therefore, to complicate the work of cost analysis; and if an attempt is made to reduce the subject to too fine a point, and too much stress laid upon it, the attention of the superintendent, foreman, subforeman, and timekeeper is soon distracted, with a consequent increase in cost. While it is an excellent thing to have costs recorded in such shape that they may be analyzed, still, if the idea is carried to the extreme, there actually results an increased payroll through lack of proper supervision and attention, and the added data will hardly make up for the loss of money.

Regarding Material. The foregoing remarks have a special reference to the cost of labor, but to a very large extent similar conditions exist with reference to the cost of material and other items entering into the cost of construction. Material delivered to the manufacturing plant can be properly stored and protected, and can be issued from a storeroom only on written order, thus keeping a perfect system of reports as to material used.

On construction work, however, such a refinement of record-keeping is absolutely impossible. Material received can, of course,

be checked up and protected to some extent, but nevertheless, all material is out in the open, subject to damage by the weather or loss by thieving. The employes have direct access to the material as it is piled around convenient to the work, and outside of supervision by the timekeeper or foreman there is only a limited check upon the material used. A carpenter will invariably saw a short piece of board from a long, new piece, rather than draw out the nails and clean up an old board which would have answered his purpose.

When the size of the gang is considerable, the irresponsibility of the character of the labor and the difficulty of watching each man all the time make it evident that the cost analysis of the material used in construction work and the cost analysis of material used in a manufacturing enterprise must be considered from different viewpoints.

Regarding Expense Items. If cost analysis in construction work is unsatisfactory so far as labor and material are concerned, how much more so is it the case in consideration of expense items—such as fittings and repairs, small tools, boots, tarpaulins, etc.—which are of just as much importance to the work as the labor and materials. These items, insignificant in character and few with reference to the unit, are in the aggregate of tremendous consequence. Small tools may be purchased, thrown down, and immediately covered by the excavated material; bolts and small tools may be stolen by the workmen; large pieces of packing, waste, or the remaining supply of bolts, nails, or other material may be thrown aside and lost. These are leaks which apparently should be overcome by the proper kind of supervision, yet if too much attention is paid to this line of the work, it is both possible and probable that the construction work itself will be neglected, resulting in a decreased output or increased cost.

Cost Analysis Requires Dollar Not Cent Policy. Cost analysis, therefore, in connection with construction work, is a subject which requires a broad-minded policy, and a system far different from the one possible in manufacturing concerns. A man who may have just the right turn of clerical mind, who may be exact in his accounts and correct in his reports as a payroll clerk, timekeeper, or stockroom clerk for a manufacturing plant, may be an absolute failure when placed in like position upon construction work.

It may seem rather a broad statement to make, but the writer is clearly of the opinion that the construction man in the field—as superintendent, foreman, material clerk, or timekeeper—must consider all accounts as they arise in the guise of dollars, and not cents. *"Penny wise and pound foolish" can have no better exemplification than in construction work, and yet cents must be considered where the cents occur in such a way as to be cumulative.*

COST ANALYSIS AS APPLIED TO LABOR COSTS

Cost Records vs. Individual Impressions. Conceding that cost analysis is fully as important to the contractor as to the manufacturer, we must make the attempt to overcome the difficulties and to obtain the true cost just as nearly as the conditions, character, and importance of the work, and the cost of securing the information, will justify.

But in so obtaining costs in construction work, there is a further difference when we compare the factory, viz: When such recorded costs do not exist, the mind of the estimator, or contractor is entirely free from any influence which might tend to make him doubt his personal recollection or knowledge of the work during the progress of construction. Where such recorded costs do exist, it is but natural that they should be referred to in making new estimates, and the result is to subordinate the personal recollection in favor of the black-and-white figures actually compiled.

If this is true of the individual who is using his own cost records, how much more is it true of the one who must base his decisions, at least to some extent, upon the recorded costs of others. Every individual who has compiled cost records and has occasion to refer to them, will, as soon as reference is made to the work, unconsciously have a mental picture of it, of all the conditions surrounding it, and of the circumstances under which the work was done; but, if the cost data is encumbered with unnecessarily complete descriptions, it would be of practically no value. It is therefore essential that any published cost records be accompanied by a brief pointed description of the kind of work—where and when done, the weather, the equipment used, the rates of wage paid, the character of labor, etc.—and even then this information should be used with discretion.

Limiting the Number of Distributions. The writer does not wish to convey the impression that he believes in a complicated and extensive set of distributions on any one job. It is better to reduce the number of classifications to the minimum, taking a general class of work, and letting all of the elements which go to make up that classification be recorded therein. By giving a sufficiently complete description of the circumstances and conditions, a proper mental picture of the work can be obtained. However, should the class of work carried under the general head be done under different conditions or methods, then separate distributions should be made.

Comparison of Methods. Suppose that we have a contract for the construction of a main sewer with branches, gradually decreasing in size from the outfall to the limit of the drainage area. (See Fig. 8, p. 49.)

Mental Conclusion Method. In the case of the "mental-analysis-only" sort of contractor, the work will be allowed to proceed under his personal direction, and the cost of the work will be determined entirely from personal observations from day to day, in connection with his book expenditures. These mental conclusions, while probably correct, are of value only to himself, and cannot be of service even to his own organization, except by word-of-mouth instruction.

Cost-Collection Method. In the case of a cost-collection system, where the costs of the various classes of work are kept separate for the entire job, the cost of a cubic yard of excavation will be determined as the total cost of this class of work divided by the number of cubic yards handled, and the same will be true of concrete, brick, pipelaying, etc.

Cost-Analysis Method. When the subject is given the proper study from the point of cost analysis, however, it will be seen that neither one of the preceding methods is the proper one.

First, the relative proportion of the sheeting and shoring of the trench to the cubic yards of earth handled, will depend upon the size and depth of the sewer.

Second, the amount of sheeting and shoring will depend upon whether the sewer is to be built of brick, concrete, or pipe, for with pipe the pipelaying can follow the excavation much more closely, thereby reducing the amount of timber work.

Third, the size of the sewer has an influence upon the cost, inasmuch as it fixes the quantities per lineal foot of trench. This, however, is not as applicable to excavation and pipelaying as it is to concrete work.

Fourth, there is a vast difference between the handling of trenches in wet and dry excavation, both of which may have occurred on the same job; if cost analysis does not distinguish the difference, the resulting average price is not a true indication of the value of the work.

Fifth, many sections of the work may be surrounded by special conditions, such as railroad crossings, interference with existing sewers, pipes, conduits, etc. Or the same job may include work in improved and unimproved streets. Unless the distinction is made in the records, the average cost will be misleading.

Sectional Divisions. The writer has found by a long personal experience that it is impossible to depend upon the average time-keeper or clerk to make the distinctions just outlined. Generally, of course, any job that is entered into has already been studied and some attention given to these points in the preparation of the estimate. Therefore, as soon as the actual construction is started, it is wise to divide the work into as many different sections as may be necessary, thereafter running such sections as though each were a separate contract or unit.

While it is true that the collection of cost information is rather indefinite and hard to obtain when a construction crew is passing from one section to the next, still there is the advantage of trying as closely as possible to secure the cost of the individual section, and a summary may always be obtained by adding the cost of all of the sections. Where no separation of the costs is made, there is no possible way of recovering the information lost. Where the costs are kept, even although in an unsatisfactory manner, some information can be gleaned from them. The totals and the averages are always available.

Same Class of Work—Same Job—Different Conditions. Sometimes on a straight run of sewer of uniform size, it may be necessary to cross an intersecting street where there are obstructions—such as street-railroad tracks, sewer pipes, gas pipes, conduits, etc.—and unless the intersecting street between street lines is considered as a

section by itself, and a persistent effort made to determine the cost of the labor in this section, the result will be an increased average cost over the entire length, a cost which would be misleading if the information was ever referred to in connection with a proposition of a like character where there was not an intersecting street.

Use of Record Charts and Tables. Emphasis has just been laid on the proper study of the layout of the work, and the consideration of sections, as being of the utmost importance for the office work of the analysis of costs. If the information has been secured in the manner suggested, it is at once available, and tables or charts may be prepared showing the cost of the same class of work in the various sections under different circumstances. These costs may not be absolute, nor is it expected that they will be used without mental reservations, but they do form the basis at least of intelligent thought and estimating. Inasmuch as it is extremely difficult to tabulate these results in a sufficiently clear manner to allow one to determine at a glance the information required, graphical charts are preferable. On such a chart can be grouped different classes of work under different conditions and different jobs under comparatively similar conditions, thus giving information of great assistance.

There is no one class of work which will not have within it certain essentials which can be directly compared with other work of like character. All of the specialties which enter into the particular job should be separated and carried as "specials". By standardizing the unit quantities and subordinating the individual idea to the general good, excellent comparative results may be obtained over a wide range of subjects. The writer's own experience as an engineer and contractor covers such classes of work as sewers, bridges, streets, railroad grading, railroad construction and ballasting, dredging, filling, concrete foundation, reinforced-concrete superstructures, and complete buildings, and yet most of this work has been handled harmoniously by the same office force and through the same organization. This has been possible mainly by the use of a standard form of time sheet (see Figs. 13, 14, and 15) which is uniform on all different classes of work, thereby making possible the transfer of a timekeeper or clerk from one job to another without any previous special instruction. If it has been found entirely possible to carry out such a plan in connection with one's individual

experience, then it should be practical to carry out the same idea with reference to the construction field as a whole.

DISTRIBUTION OF LABOR, MATERIAL, AND EXPENSE

Divisions of Labor, Material, and Expense. In the foregoing discussion we have considered labor cost principally. Cost analysis implies far more. Modern writers seem to be agreed upon the fact that cost is represented by labor plus material plus expense. This analysis is further elaborated (1) by the division of labor into "direct labor" and "indirect labor", or "productive labor" and "non-productive labor"; (2) by the division of material into that entering directly into the job and material incidental to the construction but necessary to the performance thereof; and (3) by the division of expense into expense items directly connected with the cost of production, and expense connected with the marketing or selling of the product.

There can be no argument as to the fact that these items are the primary elements in any business. However, when it comes to their application to any one particular line of business, there arise many different ideas as to method.

How Labor Should Be Distributed. *Direct Labor.* In construction work, direct labor may be considered as that which is necessary for the performance of any particular line of work after all materials have been delivered within reasonable limits of the site of the work.

Indirect Labor. Indirect labor, as its name indicates, includes such labor charges against the cost of materials, expense, supervision, etc., etc., as appear proper. For instance, a labor item representing the cost of the hauling of gravel, stone, or cement, in a concrete job, should appear as a material item; that is, the cost of such labor should be added to the cost of the material in order to determine the value of the material f.o.b. the site of the work. On the other hand, the labor items incident to the hauling of an outfit, or the moving of an outfit from job to job, the hauling of repairs and fittings, etc., are properly chargeable against the work as a whole, and therefore should appear as expense items. *It is necessary that these divisions be made, or otherwise there will be no common ground on which comparison can be made between any particular line of work under different conditions and circumstances.*

There is, however, a certain amount of indirect labor which appears directly upon the job but is of a more or less floating character. This should be distributed from day to day to different lines of work; for if it is at all necessary to the job, it is necessary to each item of work, and therefore should be properly distributed directly to the item, in a broad sense, so that any charge of indirect labor immediately upon the work may be avoided. *This applies to engineers, pumpmen, timekeepers, subforemen, and foremen.*

How Material Should Be Distributed. In construction work, there must necessarily be a certain amount of material which does not directly enter into the finished work and make a part of it, but which is necessary to its performance It would be an excellent plan if a complete record could be made, so that the amount of such incidental material used for each class of work could be determined, but all who have had experience know that in construction work it is absolutely necessary to supply a more or less varied character of timber, steel, and other material for use as emergencies arise, and that it is almost impossible to keep any record except for the amount of material delivered and the amount remaining as salvage at the end of the work.

Therefore, in the absence of any better method, it seems necessary to consider all such material as being charged directly against and as forming a part of the job charging the cost against the class of work affected, and giving a credit for any salvage.

How Expense Items Should Be Distributed. The item of expense is a very different problem. It includes not only material in the way of incidental supplies but also various matters of expense which cannot be considered as either material or labor. For instance, the labor expended for commissary quarters for the men, whether fixed, or movable, as wagons, is a direct expense chargeable against the work as a whole, and not against any one item. Again, the purchase of pipe, its installation for water supply, its removal, and the salvage on the pipe, are all expense items. As a further illustration, the bond and liability insurance on any job is a direct-expense item, its magnitude depending absolutely upon the size of the work.

Job expense items, therefore, are of two kinds: (1), those composed of material, labor, or expense which are necessary to the performance

of the work, and the amount of which is irrespective of the size of the job; (2) those composed of material, labor, or expense, which are necessary, but depend absolutely upon the amount of work to be performed.

In addition, we have indirect-expense items to be charged to each job, the amount, however, being determined by the average annual amount of business.

Whether or not it is better to distinguish between direct labor and supervising labor, is a question of individual practice. In the writer's judgment, it is well established that a certain amount of direct labor requires a certain amount of supervision, and that therefore the supervision rendered should be a part of the direct charge. To distribute these expense items in the proper way is a matter of great importance. We may have jobs running all the way from 20 per cent to 100 per cent labor, depending upon the character of the work and the conditions under which the contract is awarded.

*Items Showing Division of Expense.** It is apparent at once that on any job, items of expense should not be bunched, but should rather be divided among the various principal elements, so that one job may be compared with others and an intelligent idea formed of the comparative cost. As an illustration of such classification of expense items the following general divisions for ordinary construction work are suggested:

(1) Bond, Insurance, and Interest
(2) Petty Tools
(3) General Expense
(4) Fittings and Repairs
(5) Tools and Machinery
(6) Commissary and Commissary Supplies
(7) Fuel and Oil
(8) Special Incidental Costs

If these various divisions are determined for all jobs and established as certain percentages of the cost of labor and materials, or of either, we have a direct basis for comparison. The item, *Bond, Insurance, and Interest* needs no comment. To the item of *Petty Tools* should be charged such small tools as are easily lost, worn out, or stolen.

* For complete details, see "A Cost-Analysis System", p. 25.

To the division, *General Expense* is to be charged such items as nails, wire, bolts, and also the items of expense which go to make up the repairs to petty tools. To the item of *Fittings and Repairs* should be charged all work incident to the repairs and fitting up of tools and machinery.

To the item of *Tools and Machinery* can be charged only a fair rental value or annual depreciation for the tools and machinery employed on the job, it being expected that the *Fittings and Repairs* account, if properly kept and carried out, will keep the tools and machinery in good working condition.

The *Commissary* item should carry separate accounts for tents, tarpaulins, boots, lanterns, etc., etc., since the importance of these various items depends more or less on the character of the work, and each item is a sort of supplemental expense item. The commissary account should also take care of boarding-houses, and the expenses incident to them, when necessary.

The term *Fuel and Oil* is self-explanatory, designating the fuel and oil necessary for the production of power. If electric power is used the electric-current bills are also a proper charge to this account. *Special* items incident to any particular work should be considered as items of special nature and not allowed to affect the ordinary divisions.

Methods of Charging Expense Items. If then some such general division, not too elaborate, is made of the expense items, and their amount is determined by a percentage against the job as awhole, we have one method for their proper apportionment. Many manufacturers, contractors, and writers express a preference for the method of dividing the cost into the various unit items of work performed. But no one can claim that this is entirely satisfactory or exact. The percentage method to all practical purposes accomplishes the same end, and is simpler of application. To the manufacturer who is producing articles of uniform value, or articles of a uniform kind but varying in quality and therefore of different value, there may be a question as to the method.

In construction work, however, no two jobs are exactly alike, and they may be widely different; therefore they cannot be carried as are departments in a manufacturing plant, for this implies different classes of work under the same executive management.

These jobs may better be compared to a number of different plants established in different localities, each under its own management but reporting to a general executive head. The writer's contention has always been that the percentage method is just, even under these circumstances, for the higher the grade of work performed the greater the corresponding cost of labor, inspection, supervision, investment, etc., etc. Therefore it is perfectly fair to charge a greater proportion of the general-expense item against the higher grade of work, as is the case where the percentage system is used.

A very important point, however, and one that must not be overlooked if the percentage system is used, is determining whether the percentage shall be reckoned upon the cost of material, upon the cost of labor, or upon the sum of these two items. In many jobs the owner furnishes all the material, the contractor furnishing only the necessary labor. Again, the proposition may be one where the cost of materials is very large in comparison with the cost of the necessary labor.

Use of Percentage System. No better illustration could be made of these conditions than the cost of tracklaying in the construction of railways. In the first case, if the owner is to furnish all materials, and the contractor is to furnish only the labor, the latter's contract will run 100 per cent labor. On the other hand, if the contractor is to furnish the material, the contract will run several thousand dollars per mile, with the labor representing only a small per cent.

Therefore, if the percentage system is used for determination of the expense portion of cost, three methods may be used; but after the method of apportionment of the burden has been selected, the same method must be used for all time thereafter, or the results will be of no value for comparison with various jobs or for making new estimates. The following are the three methods:

(1) *Apportionment by percentage of the labor only.* This method is good, for the reason that the labor is a fair indication of the amount and character of the work done, and, unless the rate of pay or the class of labor is extraordinary, jobs may be directly compared on this basis. The method is recommended. However, since some labor must be directly charged to material, it will be necessary to deduct this labor before arriving at the actual amount of labor entering into the work.

(2) *Apportionment by percentage of material.* This is not a successful method for the reason that the cost of the material is no indication of the work done. The cost of the material may be standard at the point of production, but its cost to the contractor includes an additional charge for transportation and delivery. Therefore no comparison can be made between the job located close to the shipping point, or directly on a side track, and the same class of work at a considerable distance from the railroad where all material must be hauled.

(3) *Apportionment by percentage on the cost of material plus labor.* While in a great many ways this method is unsatisfactory, still it will give fairly comparative averages, and it has the advantage of adjusting and averaging to a certain extent the inconsistencies of labor and material. It has also the advantage of being the most simple of application. The various elements of expense cost must be kept separately on each job in order that in any new contract the various items of expense may be correctly combined.

A sharp distinction should be made between jobs of different character not coming under the same classification.

Method of Charging Construction Equipment. Depreciation of machinery has been handled in many different ways by many different writers. It has been apportioned as an annual depreciation and as a *fixed* percentage. In this problem, it seems to the writer, distinctions must be recognized between the business conditions of the construction plant and those of the permanent plant. In the permanent plant a piece of machinery, once installed, is used for its definite particular work until worn out unless some improvement necessitates its discard. In construction work, however, the contractor is continually buying, selling, and trading equipment. Equipment is purchased for the special requirements of a contract and is then disposed of. It is suggested, therefore, that if new machinery is purchased for construction work, a charge be made to the *Tools and Machinery* account of its second hand value, and that the difference between this sum and the original price be charged to the job. Inasmuch as the *Fittings and Repairs* account is a direct charge against any piece of work, the machinery if properly taken care of, should always be in condition to realize this second hand selling value.

What Constitutes Actual Cost. He who forms a judgment of the cost of one class of work merely by observation, by stop watch, or by similar means, may determine comparisons which are valuable in the supervision of work from day to day, but he does not form any true ideas of the actual cost of the work. The cost accountant, dealing with records which contain all items of miscellaneous or indirect labor and also non-productive time (caused by moving charges, etc), arrives at costs which may be safely used as true

COST
1. Material
    1. Actually entering into the work divided by items of different character
        1. Actual Quantities
        2. F. O. B. Price
        3. Waste
    2. Accessory or Incidental
        1. Quantity
        2. Price
        3. Salvage and Loss
    3. Labor
        1. Delivery to Job
2. Labor
    1. Direct or Productive
        1. Itemized by sections of varying character
        2. Itemized by kind of work done (Gen. Div)
        3. Special divisions of pay work
    2. Indirect or Nonproductive. To be proportioned over the various items of Productive Labor.
        1. On Job
            1. Supervision
            2. Repairs
            3. Incidentals
        2. Material Delivery (a material charge)
        3. General Supervision
    3. Yards or Shop. Direct Labor
3. General Charges or Expense. To be proportioned over cost of material or labor or their sum.
    1. Direct to Job
        1. Bond and Insurance
        2. Expense
        3. Fittings and Repairs
        4. Fuel and Oil
        5. Specials
        6. Commissary (where required)
    2. Indirect to Job
        1. Petty Tools
        2. Tools and Machinery
        3. Yard and Shops, Payroll and Expense of General Character
        4. General Office Payroll and Expense

Fig. 2 General Outline of Cost-Analysis Distributions

costs under the conditions. The effect upon the cost account of the conditions, circumstances, weather, quantities of work to be done, area covered by the work, general layout of the construction plan, etc., must be considered in the final analysis.

The various relations in cost analysis of the principal distribution items of "Material, Labor, and Expense", which have been discussed in the preceding pages are summed up in chart form in Fig. 2. This chart should be studied also in connection with Fig. 1 which shows in diagrammatic form the relations of the subject of cost analysis to the other divisions of a large engineering business.

Value of Printed Itemized Estimate Forms. The estimate form shown in Fig. 3, covers cost analysis from the position of the specialist in bridge construction, and only in minor respects can comparative suggestions be made, since, in general, the proposed

Right of Way and Temporary Bridge		
Removing Old Bridge		
Excavation	Wet ....Yds	@..............
	Dry ....Yds	@..............
Cofferdams, Sheeting, Shoring		
Foundation Piling.		
Falsework Piling . .		
Steel .	........Lbs	@..............
Placing		@..............
Lumber..............Bd. Ft		@..............
Less Salvage .		....
Erection Centers and Forms		
Concrete..............Yds .		
..............Bbls. Cement. . .		@..............
..............Yds. Gravel.		@..............
..............Yds. Stone		@..............
..............Yds Sand. .		@..............
Labor, Mixing and Placing		
Waterproofing and Drains		
Filling. . .	........Yds	@..............
Paving .	........Sq. Yds.	@..............
Sidewalks . . .	........Sq. Ft	@..............
Curbs. . .	........Lin Ft	@..............
Railing	........Lin. Ft.	@..............
Conduits, Lamps, and Wiring		
Removal of Centering .		
Superintendence. . .		
Bond, Insurance, etc.		
Moving Plant, etc.		
Equipment Depreciation . .		
Office Charges. . . .		
Total Field Cost		
Profit		
Bid About		

Fig 3 Estimate Form, as Used by a Prominent Bridge Contracting Company Compare with Fig. 2

form of estimate is in line with the charted summary, Fig. 2, but is reduced to a definite form for one line of work only.

The danger of an itemized printed estimate lies in the fact that special features pertaining to each job may be overlooked because of the habit which may be cultivated of relying upon the standard estimate form.

Analysis of Items. Comparing the two figures, we see that "Right of Way" in Fig. 3 comes in Fig. 2 under the item "Accessory or Incidental Material", and is a direct charge against the work.

Likewise, "Temporary Bridge" comes under the same head, and is also a direct charge unless salvage is considered. "Removing Old Bridge" is in the same division. All three of these items are special ones pertaining to each individual case and require a personal inspection of the site.

The cost of "Wet Excavation", "Dry Excavation", and of "Cofferdams", depends upon the character of the stream. These three items should be estimated as nearly as possible on a unit basis and if the contract is secured a careful survey of the actual quantity handled should be made from time to time for future reference.

"Piling for Foundations" is a direct charge, since this material remains in the structure. "Piling for False Work", however, is in the nature of accessory material, and for purposes of cost analysis must consequently have a separate classification.

"Reinforcement Steel" is a direct material charge and should be figured separately for each and every job. In cost analysis it should be estimated and reported on either a pound or tonnage basis.

"Lumber" is an accessory material and should always be estimated or reported on a basis of feet, board measure, or square feet of concrete surface covered. Lumber should never be carried as an element of the unit concrete cost, even in work of like character.

"Salvage" may be best estimated on the basis of percentage of lumber or cost of lumber. If possible, the lumber and labor items should be separated on center and form work, as the two have their distinct differences and such a separation may be of value where the proportions of the two classes show variation in past work.

"Cement", "Sand", "Stone", and "Gravel" are proportional to the cubic yards of concrete, but in addition they depend upon the specified mixture of concrete. The cost of these materials is the cost on the job, which is made up of a first cost of material plus the expense of delivery; this latter is a labor item but is to be carried as a separate item under Material, Fig. 2.

The labor on concrete depends upon the total yardage; and also on the available labor supply, union or nonunion, and its nationality, and the accessibility of the work. The latter is important inasmuch as it may be necessary to conduct boarding-houses or to provide transportation.

"Waterproofing," "Drains," "Filling", "Paving", "Sidewalks", "Curbs", "Railing", "Conduits", "Lamps", and "Wiring" are all direct material charges, because these things remain in the structure, and their amounts depend upon the plans and should be taken off in itemized quantities.

"Removal of Centering" is a further division of form and center work and should be kept separately.

"Superintendence", "Bond", "Insurance", and "Moving Plant" are part of the general charges against the work and also a direct charge. Other general charges of the same character are "Expense", "Fittings and Repairs", "Fuel and Oil", "Specials", and "Commissary". These latter items do not appear in the estimate form unless they are considered as a part of the various unit costs. This would not seem as definite a method as the one previously suggested.

"Equipment", "Depreciation", and "Office Charges" are general items not directly chargeable to the job; to these items should be added "Petty Tools", etc.

General Comments. This estimate is complete in that all principal items of work are stated, together with the quantities of such work and the unit prices given, but it would appear that the detail itemization of the unit price must be made on a separate sheet which may or may not be preserved In other words, it would seem that in analyzing the job with relation to the principal items of the work and the various quantities and elements entering into these items, the estimate does not furnish any data which in any way analyzes the unit cost of each separate item. Quantities entering into the various items of work are purely matters of calculation based on the plans and specifications for any piece of work. Actually they are to be analyzed only so far as to list separately the different items of work composing the main classification, but we would require that the unit cost set forth in the estimate be analyzed and given on the estimate sheet in full detail. This itemization of the unit price should be divided into material and labor and then these two divisions should be again divided into the various classes of material necessary to compose one unit of the principal item together with the necessary quantity and the price, thereby building up a unit price. In other words, such an estimate sheet would have columns for the principal items of work and other columns for the

various elements entering into the work. These elements, then, by quantity and by price, are extended to form their portion of the resulting unit price for the principal item or work. Such a method makes the estimate more certain, it gives the opportunity for locating possible increases in cost, and is the groundwork of future analyses.

The foregoing consideration of what constitutes cost analysis in its general outline may properly be followed by a detailed study of the forms and methods whereby we may obtain a complete cost-analysis system.

Illustrative Example. Cost Distribution in Connection with Power-House Contract. It may be of interest and profit to follow out the difficulties of cost-analysis labor distribution in connection with the building of a power-house structure, Fig. 4, where the excavation for the foundations was made in good concrete gravel and sand.

It should be noted that a tram track entirely encircles the building site and is closely adjacent to the trenches and forms for the foundation walls. This tram track passes under the incline track leading to the concrete mixer in the background. Tram tracks which may be easily shifted, are located in the completed portion of the excavation and are used as material-supply tracks for the concrete mixer utilizing the gravel and sand from the excavation. The gravel and sand in some instances was too fine for good concrete, and additional material in the way of broken stone was delivered along the street (at the right side of the illustration), and handled by the inclined chutes shown.

Attention is particularly called to this situation as we must here distinguish between the labor necessary to supply the concrete mixer with gravel and sand from the foundation and with stone from the outside street level, and that labor necessary to remove the excess excavation material which was handled by dumping from the incline in front of the mixer when concreting was not in operation. It cannot be expected that in a case of this kind the labor can be divided absolutely as to the various operations, but, by exercising good judgment, close approximations may be obtained so that the job may be placed on a comparable basis with other jobs of similar nature.

Careful study should be given Fig. 4, with regard to the opera-

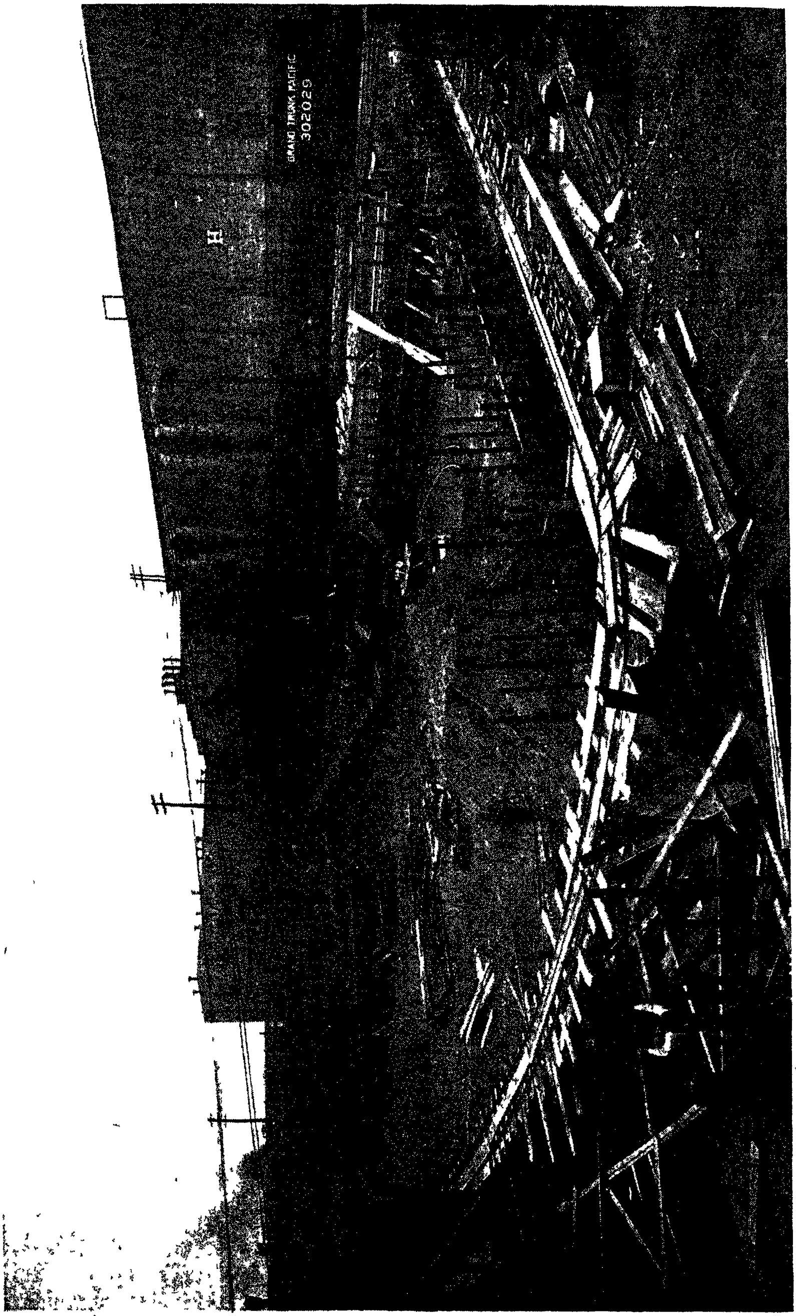


Fig 4 View of Excavation and Foundation Work for Power House Note Tram Track Completely Surrounding the Work

tions and the manner in which the work was done, to determine to what accounts the building of tram tracks and inclines, and the installing of hoisting engine, chutes, etc., should be charged.

A COST-ANALYSIS SYSTEM

Need of System. Any cost-analysis system to be of value must stand the test of actual use; therefore, a complete and logical outline of practice, from the time of preparation of an estimate on down through the actual construction and ending with the final analysis, is presented, together with explanatory forms, charts, etc.

The contractor who realizes the need of a more systematic organization, or the young man who desires to learn as quickly as possible in order to be of the most service, has need of information arranged in logical order and covering the whole subject. A multitude of books and special articles are available giving some one man's idea of some one part of an engineer's or contractor's work, such as "Office Systems", "Cost Keeping", "Field Management", etc., and yet this information, while possibly as good as the average practice, or even better, cannot be adopted, because all of these various ideas cannot be thrown indiscriminately together as a basis of a working organization, and no one idea can be adopted without requiring a correlation of all other parts of the work. As such a procedure is considered too much of a task by the average man, and as time is lacking, he generally decides to let things drift.

The writer has for a long time used in his own work a system reduced to writing, so that every man employed, no matter in what capacity, may be referred to a standard practice—and not be dependent on verbal instructions repeated time and time again. We have in consequence secured first, a closer harmony and a greater output; and, second, a larger opportunity and greater encouragement for suggestions and improvements.

We are indebted to numerous writers for a systematic study of industrial organizations and of efficiency in shop management. Some have carried the same idea and same line of thought into the engineering and construction field, and yet, in the opinion of the writer, neither the so-called cost-data motion-study system, nor any of the numerous papers printed on this subject satisfies the general

need for a practical outline and a comprehensive study of the whole subject of estimating and cost-analysis engineering.

The methods here suggested may not be the best, but they are offered as the result of practical experience of many years, and should at least be of value in forming the nucleus of any system. Certain fundamental truths exist in all contract work and these facts can be learned only by experience. For this latter reason solely, if for no other, this series of articles should be worth while, especially to the young engineer and contractor.

Particular Importance in Contracting Work. There is no doubt of the importance of systematic cost keeping, cost management, and cost analysis in industrial plants. But how much greater is their importance in the contracting field, where work of a distinctly industrial character is complicated by the fact that the operations are unconfined. In other words, the contractor's work is truly industrial, but he is handicapped by the nature of the operations, inasmuch as he must work in the open, unprotected from the elements; must attempt to keep records of men and materials which are shifting and difficult to locate; and, in general, must organize an ignorant carefree low-grade force of laborers in such a manner as to secure adequate working results; and finally, must obtain such data as will make the special job a record of permanent value.

Science of Management. In construction work the whole field of operations must be considered, for the same laborer may be moved from one class of work to another, even during a single day, and perhaps several shifts may be necessary. Along with these difficulties run the trials with union labor and its exacting stipulations as to rates of pay, hours of work, etc. It is all very well to talk about what could be done if all these obstacles were removed, but in ordinary work—the kinds of jobs we all know—these obstacles are not removed. Ordinary circumstances and conditions must govern and any system adopted must recognize them and adjust itself accordingly.

It is proper to talk of "staff" vs. "line" and to encourage the civil instead of the military type of organization, but after all, in ordinary construction work, the laborer must be driven by expert supervision and the results lie with the commander rather than with the individual workman; this is so because of the character of the

labor available. Under such conditions, all that can be expected is a fairly accurate checking of the labor employed. Such cost data is not absolute in fact, but simply characteristic of the kind of work, and sufficiently close for guidance in analysis of like work. For these reasons, cost data, so styled, can be of no value so far as assuming to report facts or truths, but it can be made of greater or less reference value, depending upon the care taken in gathering information.

Construction never can be an exact science or subject to exact rules, all contrary writings as to principles notwithstanding. There are no five, ten, or twenty principles involved. Construction, like engineering—or even more so—is a matter of common sense or good judgment, but an understanding of it may be developed and a mental analysis encouraged by a proper use of methods, standard forms, printed records, and analyses. In other words, while preaching modern methods, while recognizing all that has been written as to office and field systems, and while advocating some such system as will be outlined hereafter, *still the writer desires to emphasize the facts that these devices are of use only as developing mental capacity for accurate analysis, and that records in themselves are of questionable value.* The engineer and the contractor who work the hardest for a system and adequate records do not profit as much by the actual records as they do by the development within their own brain cells. *Do not overlook this point.* Records are valuable only in their effect on the judgment, which indicates that the method of treatment of a new proposition is founded on experience. Experience, however, may be fortified by a proper system of dividing, itemizing, and recording costs of labor and materials. Before discussing the points of the proposed system, it may be well to study the outline of the system, and the relation of the different groups to each other.

DETAIL OUTLINE FOR STUDY

Scope of Outline. Our study, then, will cover the items in the following detailed outline of work, giving prominence to those features which are most closely related to cost analysis, as shown by Fig. 1.

Group 1. Estimating the Job. Estimating the job on a basis of a definite well-considered plan, with reference to execution and definiteness in final analysis of results, involves:

(1) The examination of specifications for unusual clauses, and the preparation of abstracts. See Figs. 5 and 6.

(2) The comparison of plans and specifications, making notes of special and peculiar features. See Figs. 5 and 6.

(3) A proper method of taking off quantities, including checking. See Fig. 13.

(4) The form of estimating blanks. See Figs. 14 and 15.

(5) The layout of work and method of operation. See Figs. 8 and 11.

(6) The division of work. See Figs. 7 and 12.

(7) The use of short cuts and approximations in checking.

(8) Consideration as to when quantities are important and when unit prices are important.

(9) The estimating of labor and material separately.

(10) The estimating of general charges, and how general charges are affected by character of work. See Tables II and III, Part II.

(11) Determination of equipment required, and how it is to be charged out.

(12) Preparation or notes and sketches showing basis of estimate.

(13) The filing of estimates for future reference.

(14) The value of estimates regardless of execution of work.

(15) The preparation of a personal "dope" book.

Group II. Construction of Job. The construction of the job following as closely as possible the estimated plan of operation and, where that is defective and requires modification and change, revising or extending estimate to cover the changes, involves:

(1) Selection of equipment.

(2) Arrangement of the organization and distribution of forces.

(3) Preparation of instructions to timekeepers and field forces.

(4) Preparation of progress plans and instructions to the field engineers. See Figs. 10 and 12.

(5) Preparation of forms for quantity and cost reports. See Figs. 13, 14, and 15.

(6) Division of work by sections. See Figs. 7, 8 and 11.

(7) Selection of units with regard for simplicity of items, and with essentials separated.

(8) Designation as to direct and indirect labor and materials. See Fig. 2, p. 19.

(9) Selection of field approximations or unit measurements for reports.

(10) Supervision over the general office system of purchasing.

(11) Instructions as to the distribution of cost of material and labor and the making of monthly estimates.

(12) Instructions as to better supervision by use of the daily reports.

(13) Specific instructions to (1) foremen, (2) timekeepers and clerks, (3) engineers and firemen, (4) mechanics, (5) yardmen, (6) general office men, and (7) for weekly cabinet meetings.

Group III. Analysis of Job. This means carrying out the work so far as possible according to estimate, but with due allowance for actual reports based on conditions as found during execution, and requires:

(1) Maintaining cost, quantities, and unit costs strictly up to date. See Figs. 13, 14, and 15.

(2) A checking of rate of progress. Figs. 10 and 12.

(3) Revision of original estimates from time to time.

(4) Report of discovery of weak points. See progress charts, Figs. 26 and 27.

(5) Report of daily status of work. See Figs. 26 and 27.

(6) Report of revised estimate as to time of completion. See Figs. 26 and 27.

(7) Report of comparisons by selected sections.

(8) Report of efficiency of men and equipment.

(9) Report of comparison with other jobs of like character.

(10) A careful separation of ordinary from unusual units.

(11) The final form for tabulation of results.

(12) The final form of graphical chart.

(13) Notations that results are not absolute but to encourage mental discipline.

(14) Care in the copying of data.

(15) Determination of production and establishment of a standard rate of pay where possible.

(16) Determination of cost of materials f.o b. job and comparison with purchase price

(17) Caution as to the use of stop-watch methods.

(18) Notations as to padded pay rolls.

There may be other ways or methods as good or better than those suggested; any system is good that recognizes the close relationship existing between estimating, construction, and final analysis.

Value of Estimates. A carefully prepared estimate on work not secured is not valueless but should be preserved and indexed to form a part of the personal files of the contractor or estimator. In addition these estimates should be summarized and compared with others and with those estimates where the job has been actually constructed. Such a comparison cannot help but be of benefit; first, by emphasizing the fact that the characteristics and individuality of every job require special consideration, and so disabusing the mind of the idea that any fixed or lump-sum unit prices can be used indiscriminately; second, by the development of a greater estimating ability and by the correction of preconceived ideas of estimating, which will be modified by studying those estimates which have gone through actual construction. A comparison of this kind is just as properly a part of analysis as the cost records of any particular job and, further, if this plan is carried out, the tendency is to build up the cost-analysis turn of mind, so that it becomes almost instinctive.

Suppose the job is secured, then the estimate, if it has been made in detail, is of still more value for future comparison, since the opportunity is afforded to check the separate items during the progress of the construction. The resulting final cost analysis is truly an analysis, rather than a mere collection of cost records In other words, the estimate may be properly designated as a preliminary plan of attack, to be followed by the filling in of details secured during construction, until the completed report, developed through the final analysis, is eventually made.

Analysis of Subdivisions. Before entering upon the detailed discussion of methods under the divisions suggested, it is probably well to set forth in a condensed form the logical steps by which the work advances from the time of the estimate until the time of the final analysis. These steps, of course, are included under the

primary divisions of estimating, construction, and analysis. In order that the student may have before him at the beginning in a condensed form the idea of systematic sequence of thought and operation, the following program is suggested:

Estimating—
(1) Outline of work.
(2) Scheme of operations.
(3) Itemized quantity estimates.
(4) Itemized cost estimates.

Construction—
(5) Labor cost distribution.
(6) Progress by sections.
(7) Labor-cost bookkeeping.
(8) Material-cost bookkeeping.

Analysis—
(9) Analytical progress estimates.
(10) Analytical comparisons.
(11) Progress charts.
(12) Final cost analysis.

It will be noted that by this program the work proceeds in natural order from beginning to end.

In Fig. 1 under Group 1, we have "Estimates, Construction, and Analysis", each of which we have now dissected into four parts. In Group 2 we have the three elements comprising the complete cost, the details of which have been previously discussed and finally presented in condensed form in the chart, Fig. 2. In Group 3 we find the means whereby the elements of cost are so digested that we secure the data for the final cost analysis.

The divisions under the General Principals, Group 1, are evident. On the left hand in Group 2 we have the physical forces to be handled and controlled, and on the right hand in Group 3 we have the forces with which this result is to be accomplished. These secondary or physical forces and tertiary or operative forces may now be subdivided into items covering all the operations, methods, forms of blanks, reports and everything that goes to make up a complete cost-analysis organization.

Each one of the Principals, Group 1, is directly concerned with each one of the three divisions under Physicals, Group 2, and Operatives, Group 3, and in addition the Physicals and Operatives are dependent one upon the other. It requires only a moment's thought to realize that every line connecting these various divisions represents a distinct systematic line of thought, and that these Principal connectives represent numerous different viewpoints, which in the aggregate we may term *mental analysis*.

This is not in any sense complex, and in reality is no more than the ordinary unconscious working of the mind of the successful contractor. It is not necessary that any office be departmentalized and conducted on an elaborate scale. Every one of the operative divisions can, in minor practice, be actually conducted and performed by the engineer or contractor individually and the same results in system will be accomplished. Such a contractor will be his own engineer in the planning and estimating of the work; he may also be his own superintendent and foreman during construction, and he is certainly going to be his own analyst when the work is completed.

The "Estimate, Construction, and Analysis" (Group 1), must each consider "Material, Labor, and General Charges" (Group 2), but each from a different viewpoint, viz, future, present, or past. Similarly Group 3, "Engineering, Bookkeeping and Clerical Work" involves Group 1 and Group 2. Take the last division of the Operatives—Clerical. If properly conducted, this division must consider the original Estimate, the actual Construction, and the final Analysis and must compare them. It must consider "Material, Labor, and General Charges" with reference to what was estimated and what was actually used, and analyze the differences; it itself is also directly dependent upon the Engineering and Bookkeeping records in its own Operative class for a consistent, harmonious, co-operative working. We are using in this article the term Engineering to refer to any method whereby reports as to the quantity of work done or estimated are obtained; and the term Bookkeeping to refer to the distribution of money actually expended. The Clerical organization is therefore, for the purpose of this article, the analysis man, although in actual practice the analysis may devolve upon the engineer, the bookkeeper, or the contractor himself.

The Physicals would seem to be easy of separation and directly dependent upon the work to be estimated, but it is only by the closest possible unification of action of the Operatives that this result is actually accomplished. Material, as before mentioned, sometimes includes labor, inasmuch as the material charge against the work is the entire cost delivered on the job.

It would be wrong to include such a labor charge in the labor-cost distribution of handling this material at the job. Two jobs exactly alike, where the labor distribution would vary but slightly, may be so located that the labor cost of delivery of materials to the one job, located in a city or on a construction railroad siding, is nothing, whereas cost of delivery to the other job, located at some distance from a track may double the material cost. Labor sometimes includes material or general-charge items—as, for instance, where a subcontract is made for the furnishing of labor, but where the subcontractor furnishes all or part of the materials or supplies necessary in the handling of his class of work.

It therefore rests with the cost-analysis engineer so to train the various employes that they will have constantly in mind the principal divisions of cost engineering and will facilitate the analysis by carefully watching and separating and distributing the cost of material and labor and general charges. When this is done then the everyday work is more satisfactory, profits are increased during the construction period, and future estimates are more reliable.

DETAILED STUDY OF FORMS AND MATERIALS

OUTLINE OF WORK

"Outline of Work" Form Standard for All Jobs. As soon as the plans and specifications for any new job are received in the office the first step is to go through the specifications, noting each requirement, using the standard form, "Outline of Work", illustrated in Figs. 5 and 6.

This form should be a standard and is to be used for every job. As a result of practical experience, the form has been found to cover all working conditions, whether the job is large or small. It can be used for bridges, buildings, sewers, earth work, railway construction, pavements, sidewalks, and, in fact, for every kind of work coming to the engineer or contractor.

Discussion of General Headings. The benefits to be obtained from the use of this standard form may be briefly stated as follows:

(1) In filling in the general heading, Figs. 5 and 6, giving the nature of the work, where located, owner, and architect or engineer, together with the estimate number, we secure an index for permanent record. It is suggested that estimate numbers begin with No. 1 each year, since the estimating labor for each season is thus automatically recorded. Knowing the number of jobs secured and being careful to distribute the cost of labor in estimating all jobs, we may arrive at a percentage of cost that such estimate bears to the total cost of our general office charges. Such general office charges should preferably be kept separately for each calendar year, together with the total volume of business for the same period.

When times are dull and competition is intense, then estimating will constitute a larger percentage of these charges than when work is plentiful and most of the general office time is spent on actual supervision. This overhead charge, as a whole, naturally fluctuates, since it is considered as a percentage on the total volume of business. Rent, light, heat, etc., together with a certain amount of salaries (which must be maintained regardless of new work) are all independent of the volume of business, and so the percentage to be allowed must really be a matter of good judgment based on extended past experience. In construction work, one cannot count on a steady increase or increment each year, since, notwithstanding its reputation, a company must compete by bidding for every job secured. Therefore, not to burden any one job when times are dull, a reasonable amount should be added to each estimate throughout the year for just such charges. An average over several years is, of course, of more value than the actual charges for any one period. The cost of estimating should also be determined as a percentage of the total estimated cost of proposed work whether the jobs are or are not secured. Such records give data upon which to base a proposition when one is called upon for examination of a promotion project. From an engineering and estimation standpoint, regardless of construction, costs of surveys, reports, etc., should be kept, since many times propositions will arise which are to be used for promotion purposes and should be paid for by the parties benefited.

(2) *Discussion of Secondary Headings.* In filling in the sec-

ondary headings with date of letting, name of party or parties to whom the bid is to be addressed, amount of certified check, amount of bond, form to be used, amount and manner of payments during progress, beginning and completion of the work, and penalty and bonus stipulations, we have all the essentials necessary for the submission of a formal proposition, with due consideration, also, for the financing of the work. The latter point is of great importance, since many times the payments, the penalty, and the bonus must be considered as a part of the cost of the work. For instance, in some cases weekly estimates are allowed the contractor, and a job of this kind will finance itself. (Incidentally, it may also be remarked that under such terms—at least not exceeding monthly estimates—the owner secures his work at the minimum of cost.) In other work, especially in that for some municipalities, no payments are made until the work is completed and accepted. In a case of this kind arrangements must be made to secure funds to carry the entire cost, and there must be included in the estimate the interest on such funds. As the loaning of money on contract work in these circumstances must be considered as a speculation rather than an investment, money probably cannot be secured for better than the usual rate of interest plus a bonus, and in the aggregate these two items will amount to at least 10 or 12 per cent. Under such conditions either the owner or the public pays the bill, provided the bid is properly made.

Sometimes, when the time of completion is the principal element in awarding the contract, it is wise to include in the cost an allowance for anticipated penalty, and to set the time for completion at a very early date. On the other hand, if a bonus is offered, it is probably advisable to set an early date for completion and to make a corresponding increase in the unit cost of the work, and to look to the bonus to repay the extra cost and decreased output caused by overtime and night work. (See description of work shown in Figs. 11 and 12.)

(3) *Abstract of Specifications and Plans.* The third heading covers the abstract of the specifications and plans, and serves the purpose of itemizing the principal work with a reference to the appropriate page of the specifications and the correct sheet of plans. The larger space is given to notations with regard to general require-

ments, and a secondary space is allowed for notations of special and unusual features. This arrangement shows instantly whether the specifications and plans are normal or abnormal. The fewer the items that appear in the special column, the greater certainty there will be in estimating.

By allowing a final column for reference to the page of the estimate we not only have the reference, but also the opportunity for checking off each item so as to be sure nothing is forgotten. A notation should be made as to who prepares such an outline or abstract of work, and in case of important work it should be checked. The benefit of such an outline of work is not confined to the foregoing suggestions, but also exists in the more systematic, intimate acquaintance with the subject matter gained by the one making the abstract and checking it.

In any case in making a proposal we read the specifications and look over the plans. If at the time of doing this a blank form is used, as suggested, we not only guard against omissions but by concentrating our mind to produce the concise, condensed abstract, we gain a more lasting recollection of the requirements. We are also prompted to differentiate between the normal and abnormal requirements. It takes but a few minutes longer to go through a set of plans and specifications in this way, and the result is far more satisfactory, aside from the fact that such abstracts, whether the job is secured or not, are of permanent value as a part of the estimate files.

Such an actual summary of work taken from the individual specifications is preferable to any standard printed form (see Fig. 3), even if it be a specialist who attempts to list everything on a standard estimate sheet. Human nature will prompt a close adherence to such a form and a slighting of specials pertaining to the particular job. Since the printed form, moreover, is arranged in a certain order with very little opportunity for interpolations and additions, the tendency is to condense items or to run specials under "miscellaneous" or "incidentals", which misleads and makes the estimate obscure.

Comparisons of Outlines, Figs. 5 and 6. As an illustration, Fig. 5 shows a standard form of this kind used in estimating a reinforced-concrete bridge. The actual size of the blank used for this

work was $8\frac{1}{2} \times 14$ inches, and the abstract covers but 1 page. The original specifications for this work covered over 18 pages of the same size, and yet on this abstract are contained all the essentials for estimating, together with page references to the specifications and plans for any required details. This is one of the cases where the specifications themselves are not indexed or paged, a condition which is more often the case than would be expected. The "Outline of Work" supplies the deficiencies of the specifications, and at the same time presents the matter to the contractor in the best possible condensed form for pricing. Special attention is called to the many notations in the "Special and Unusual Features" column. Most of these items cannot help but have an effect upon the cost because of the uncertainty as to the meaning of the specifications and as to the requirements made upon the engineer.

Compare the use of Fig. 5 with Fig. 6, which is the same standard form but covers a reinforced-concrete building. Note how few items appear in the special column. This at once is an indication that the specifications for the building are definite and there is no occasion for a misunderstanding. Both Figs. 5 and 6 are bona fide abstracts of actual specifications purposely used as illustrations for the reason that only by using such actual cases can we show the adaptability of the system to the various sorts of plans and specifications different architects and engineers may prepare. This is not a criticism of the architect or engineer. The minds of all men do not work alike; some prepare strong specifications, with an absolute lack of system in indexing, arrangement, and definiteness; others are extremely systematic, but produce specifications which are unintelligible or inconsistent.

Comparison of Plans and Specifications. The next thing in order is carefully to inspect the plans, and compare them with the abstract of specifications. Make such explanatory notes on the abstract as the plans suggest, and add to the abstract any additional work shown on the plans not specifically mentioned in the specifications. Compare the specifications and plans and note discrepancies with a view to securing explanations from the architect or engineer; failing in this be sure the estimate is on the safe side.

Scheme of Handling. Logically the next step before taking off any quantities is to decide, carefully and without haste, how the

job under consideration can be handled and just what equipment and what organization are necessary for the most economical execution of the work.

This first decision is all important. Many times a "guess bid" is made without any particular thought as to how the job is to be handled if the bid is successful. This all-important decision is deferred until time demonstrates whether or not the work is to be actually carried through. Such a method is foolish.

An intelligent estimate can be made only after a proper consideration of the questions of organization, equipment, and method of operation—the final proposal being then made with a confidence that the figures submitted are reasonable and accurate, and with the expectation of being awarded the contract.

No proposal submitted can result in any satisfaction to either the recipient or the contractor which does not include such a careful examination of the plans and specifications and an inspection of the site of the work. If this study is given the proposition there will result a plan of operation and a feeling of certainty and promptness in the beginning of the work which will go a long way toward satisfying both parties.

Changes in this preliminary plan may be required, and new ideas and methods adopted during the actual progress of the contract, but these in no way affect the definite plan which must be the basis of the estimate in the beginning of the work. Any successful proposal requires a definite outline of action and it is better to determine this at the time of making the estimate than to await the awarding of the contract. Any estimate lacking such a plan of operation will not bear investigation or examination.

Set of Preliminary Plans of Work. Granting, then, that the plan of attack has been decided upon, the decision should be made a matter of record, both by written description and by the preparation of sketch plans showing the general layout of the plant and method of operation. These plans serve as a basis of the amount to be included in the estimate for equipment and the labor of installation, and, in case the job is secured, as a reminder and guide in the actual moving in on the work.

It is true that many times improvements are made on the preliminary plans, but in the majority of cases such changes are

prompted by the study of the proposition at the time the estimate was made. There can be no improvement or better second-thought unless at some time previous suggestions have been made. Prepare this preliminary plan in such a form that it may be preserved. If possible, make it on the same scale as the general plans of the work. In a great many cases this can be accomplished by making a tracing or sketch with the addition of such notations and comments as are necessary to explain the estimate. If, however, the work covers a great many sheets and is thereby complicated, it is best to prepare a new plan on a reduced scale, indicating all the work to be done. A condensed profile will assist the preparation of an estimate and its value be far in excess of the value of the time taken to prepare it.

It is astonishing in how many cases a really complete set of plans from an engineering and architectural viewpoint is lacking in this condensed information required for inspection and an intelligent estimate by the construction man. In one day the writer condensed on a sheet of double legal size (17×14 inches) all the essentials in plan, profile, and section of a sewer job composing 10 sheets, 24×36 inches. The sheet thus prepared did not displace these working drawings, but it did furnish a "bird's-eye view" of the entire job, with which the engineer could go into the field, view the site, and be properly prepared to make an intelligent estimate from the working drawings. This condensed sheet, giving full information, was prepared when the writer was called as a construction expert in litigation over a certain sewer which, it was contended, was deficient in many respects. During the trial it developed that all the available references were to this exhaustive report and set of working drawings. No ordinary man could be expected to ascertain in reasonable time to what particular part of the work a witness' testimony referred. The simple sketch-map referred to served the purpose of fixing, in the minds of all interested, the direction of the sewer and the relation to streets occupied and to cross streets, and designated without any possible misunderstanding the location of all troubles testified to.

Illustrative Examples. 1. A similar condensed layout is shown in Figs. 7, 8, and 9. The labor spent in preparing an outline of this sort is not lost, inasmuch as it forms the beginning of just such a

preliminary operation and layout plan as mentioned above. And this is not all, for if such a plan is properly prepared it will take its place, in case the job is secured, in the system of progress reports. For the purpose of estimating, the plan may be prepared roughly as a sketch, and in case the proposal is accepted tracings can be

Section	Location: Interi'n	Location: Between		Type: Con	Type: P	L'gth	Excavation: Av Cut	Excavation Cubic Yards: Est.	Actual	Surp	Gravel-Clay	Wet Dry	Concrete: Yds. Per Foot	Total	Centers	Type of Mach: Excav	Concrete	Progress Date: Beg	Com	Tot. Days	Av Ft Day	Foreman	Perct	Remarks
A	Fleming		29th	6'-0"		1265	8	3186		2024			519	657										
A1	Fleming		Ruckle		24	350	12	454		67														
B	Ruckle	Flem	30th	3'-0"		1310	9	2402		747			.285	373										
B1		30th	32nd	2'6"		1345	9	2194		592			.248	334										
B2		32nd	33rd		20	665	16	1380		126														
B3		33rd	34th		18	645	16	1147		105														
C	Park	Flem	30th	3'-0"		1150	10	2343		656			.285	328										
C1		30th	32nd	2-6"		1340	9	2233		590			.248	332										
C2		32nd	34th		20	1320	8	1332		251														
D	Brdwy.	29th	30th	3'-0"		670	9	1228		382			.285	191										
D1		30th	32nd	2'-6"		1350	10	2500		594			.248	336										
D2		32nd	34th		20	1315	8	1332		250														
E	College	Brdy	32nd	3'0"		2325		4736					285	663										
F	32nd	Coll		5'0"		400	12	1278					441	176										
G	College	32nd	38th	6'0"		3940	13	16125		6304			519	2045										
H	38th	Coll.	Cent.	8'-0"		1325	24	12956		3678			1.048	1389										
H1		"		5'0"		65	23	416					441	29										
H2		Cent	Penn	6'-0"		1325	21	8743		2120			.519	688										
H3		Penn	Ill.	5'-0"		1325	19	6993		1590			.441	584										
H4		Ill.	Sen.	4'0"		1335	17	5464		1335			.363	484										
H5		Sen	Chas.	3'3"		1005	18	5852		603			.304	306										
H6		Chas.	Byram	3'-0"		510	14	1452		291			.285	145										
H7		Byram	Consr		24	500	15	973		95														
I	Central	38th			15	345	14	540																
I1					12	345	14	447																
J	Washington Blvd	38th			18	690	16	1432		100														
J1					12	700	12	404																
K	Meridan	38th	40th		20	1330	23	3965		130														
K1		40th			18	1270	22	3105		100														
K2			42nd		15	600	13	866																
L	Kenwood	38th	40th		20	1285	12	2000		125														
M	Capitol	38th	39th		20	570	14	1000																
M1		39th	40th		18	715	14	1113																
N	Graceland	38th	40th		18	1285	14	2053		100														

Fig 7 Outline and Progress Chart of Sewer Job Shown in Plan Form in Fig 8

made and blue prints secured which will serve to illustrate in the best possible manner the field reports.

Fig. 8 illustrates a sewer job for which there was prepared a "Scheme of Operation", shown in Fig. 7, and a profile, Fig. 9, of the work contemplated. This first rough sketch was used in inspecting the site, in making notes and in planning the organization and mode of operation. In its preliminary stage the sketch was

not nearly so elaborate, as shown in the plate, but it answered the needs of the moment. When the job was secured the sketch was completed and traced in a neat and complete shape, as shown, to

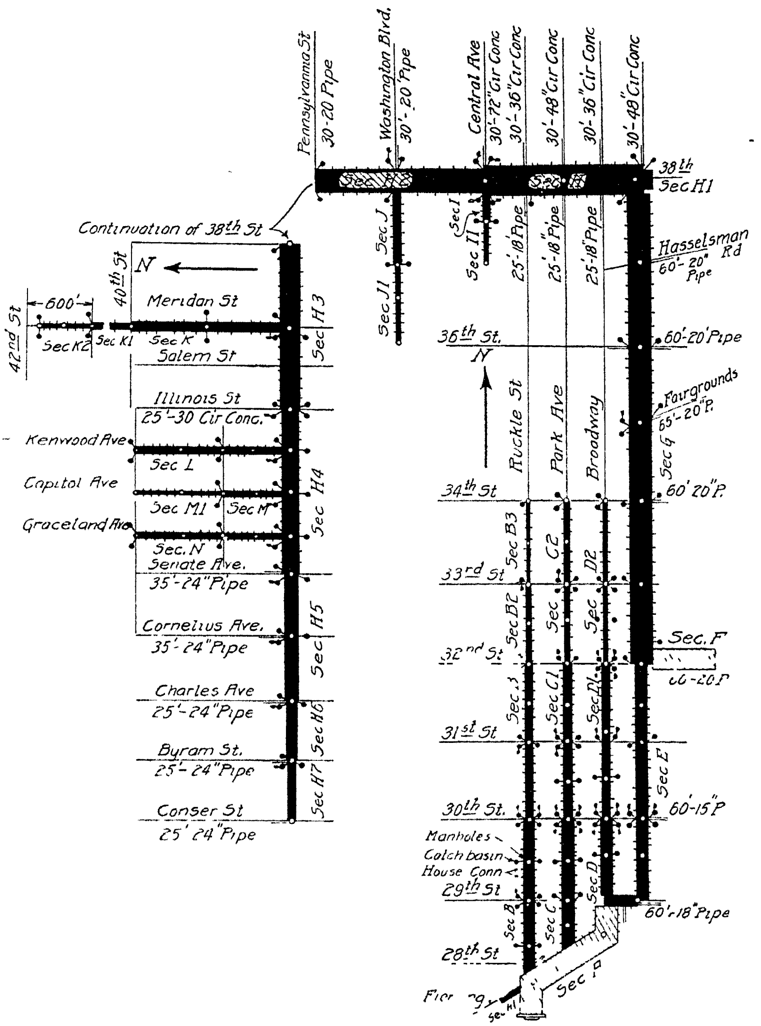


Fig 8 Layout for College Avenue Sewer Contract For Details of Section A See Fig 10

form the basis of all working thereafter. Additions were made to harmonize with the progress charts, as will be explained later.

Object of Having Sections. It will be noted that this sewer

job, Fig. 8, was cut up into 34 sections for the purpose of progress charts, reports, and cost analysis. By a reference to the table, Fig. 7, the need of such a method is suggested, not only for this particular job, but for any work.

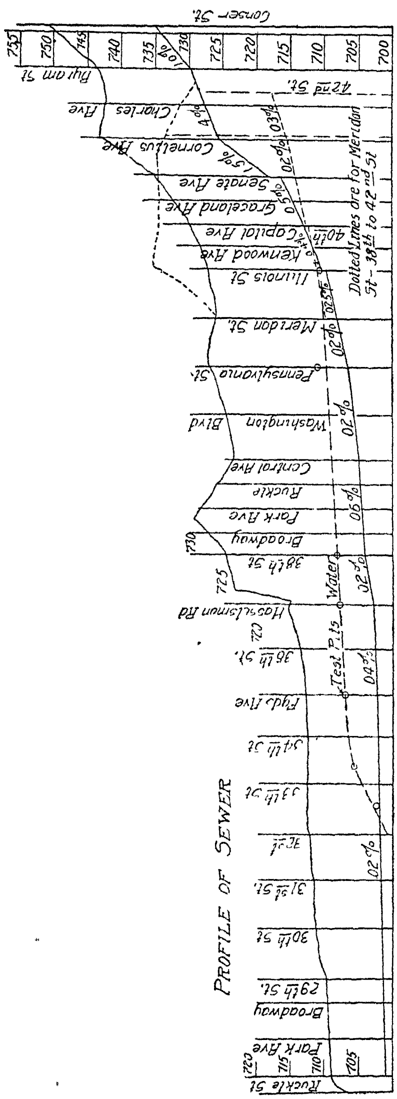


Fig. 9. Profile of Sewer Shown in Plan in Fig 8

For example, Section A, Fig. 8, is a 6-foot circular concrete sewer in an average cut of 8 feet (see Fig. 10). Section H, Fig. 8, is an 8-foot circular concrete sewer in a cut of 24 feet. Both of these sections are a part of the same sewer system, but certainly it is not to be expected that either the cost of excavation or of concrete will be the same or even approximately so. Any average over the whole job—including these sections and the others of intermediate sizes and depths as well as pipe sewers—would be misleading for future reference and estimates. Within the limits of this job are contained a great number of different sizes of sewer, both concrete and pipe, to be constructed at various depths, and some of the same size but with construction at different depths. For example, Section A, Fig. 8, calls for an average cut of 8 feet, requiring only occasional shoring of banks and no solid sheeting. Section H2 calls for a 21-foot cut, requiring solid sheeting in two sections of approximately 12 feet each. While the sizes of sewer are the same,

in the two instances, it is not to be expected that the cost of excavation can be the same, nor will the cost of the concrete be the same, as the rate of progress directly affects this cost.

Again, Section H, an 8-foot concrete sewer, with 12-inch walls, requires a trench of a bottom width of 10 feet. Compare this with Section K, at the top left hand, of Fig. 8, which is practically of the same depth but has only a 20-inch pipe, requiring a bottom width of trench of, say, 3 feet. The first section involves a large yardage per lineal foot and the latter a small yardage, yet both require practically the same amount of sheeting lumber, and labor for placing and drawing it. Can the yardage labor cost be the same? No average cost of labor is applicable to any one section; therefore, to gain an intelligent record for future use where these sizes and depths may occur in different propositions, it is necessary to keep the cost by dividing the work into sections. *Other things than those mentioned make the*

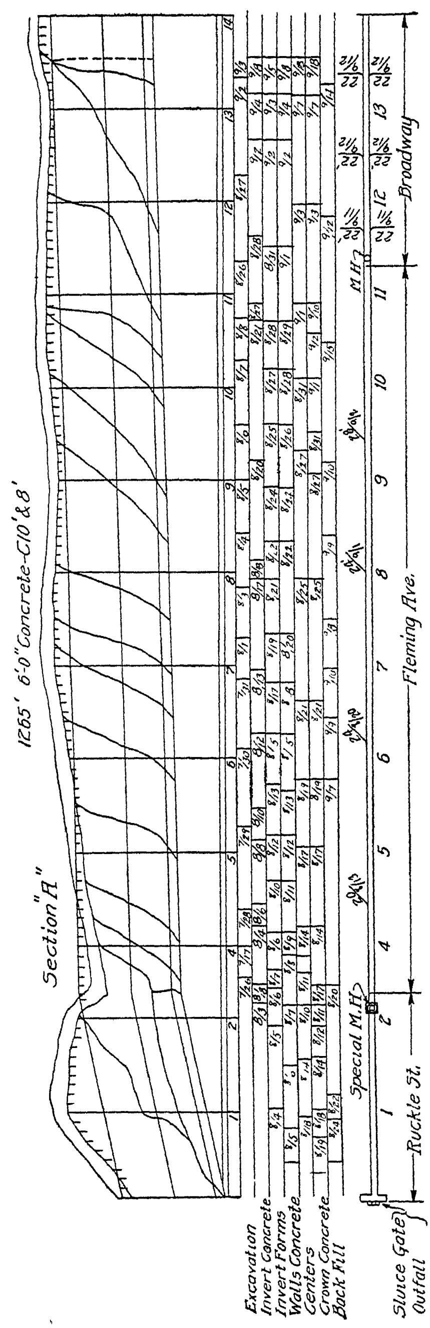


Fig. 10. Detailed Progress Chart for Concrete Sewer Shown as Section A in Fig 8

division by sections the most important of all stipulations as to estimate, construction, and analysis.

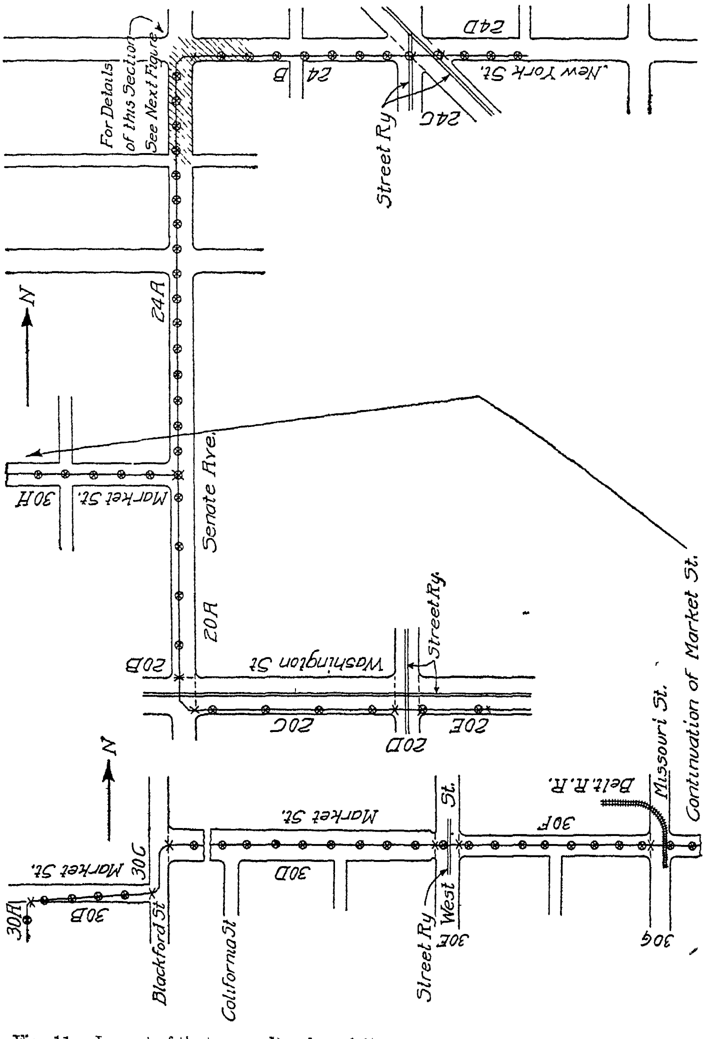


Fig. 11. Layout of Section on Reinforced Concrete Conduit For Details of Shaded Portion See Fig 12

2. Work in which there is no large variation of quantities may be constructed under absolutely different conditions within the limits of the same job. Figs. 11 and 12 show a situation of this kind. This work consisted of a reinforced-concrete conduit

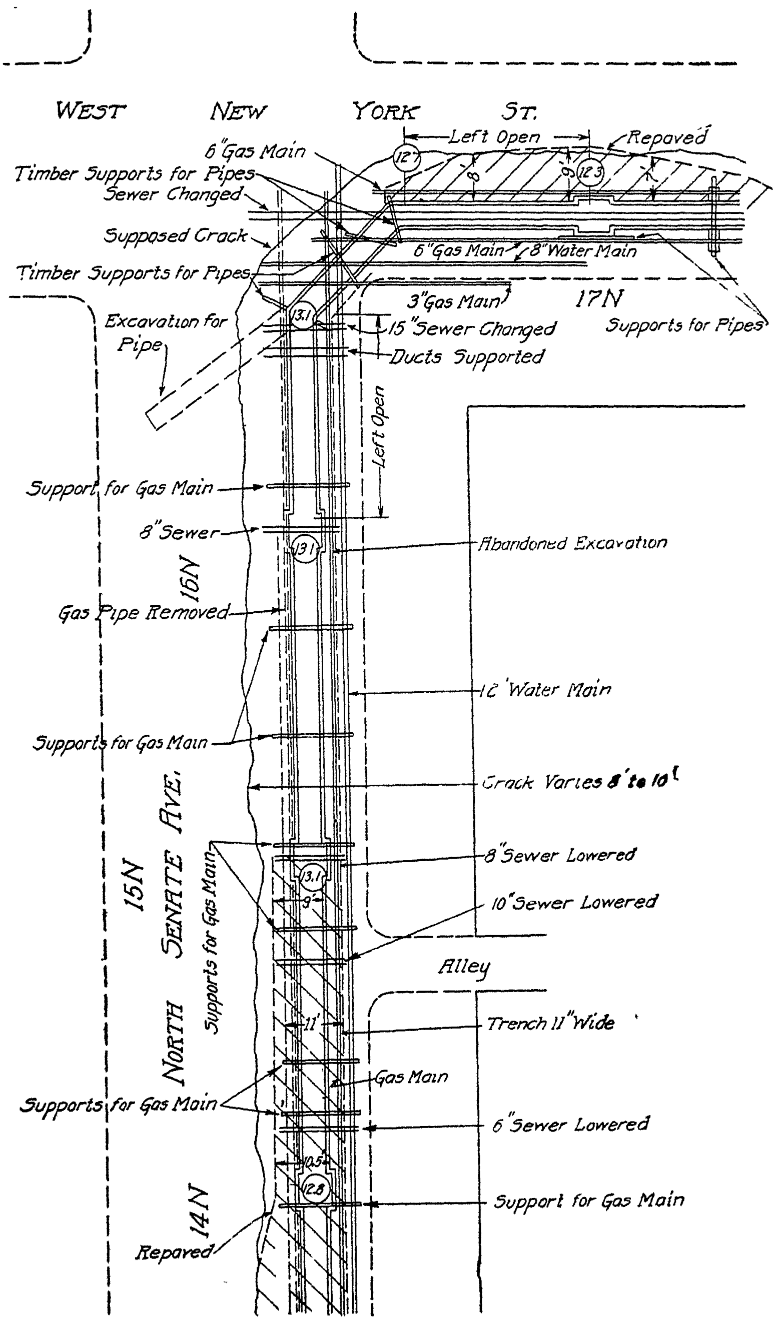


Fig 12. Details of Reinforced Concrete Conduit Shown in Fig 11 Enlarged View Shown to Indicate Complications of Water and Gas Mains and Crossings

in the down-town streets of a large city. It will be noted that the job is divided into sixteen sections, although there are only three sizes of conduits and the depth of excavation is really uniform. In this case the prefix numerals in each section indicate the size of pipe of the conduit. Section 24-B cannot be compared with Section 24-C, because the latter work is complicated by a street crossing, involving a diverging double-track trunk-line street railway and requiring tunneling under four tracks and special track work.

Similar classifications should be made on all construction work—bridges, buildings, and what not. Later a more extended study will be given this idea, which is, as we have said before, the key to all successful analysis, more important even than any system of cost recording, cost keeping, or any form of blanks used.

Averages are valuable when properly prepared but "guess" averages are dangerous, especially so when they are based on so-called cost records which cover a whole job without a recognition of this point of sectional division.

Lump-sum unit costs are more valuable when proper consideration is given to sectional division than are the most exact itemized unit costs when no attempt is made to segregate.

ITEMIZED QUANTITY ESTIMATES

Checking Quantities. Carefulness and methodical attention to detail are much more important in taking off quantities than in setting the price for the individual kinds of work. The quantities, in their various proportions exist for one job and one job only, and there is nothing in past experience to guide the judgment or form a mental check. The quantities of the different kinds of work when set forth on an estimate sheet must be accepted as correct, but are correct just so far as the estimate has been carefully prepared by a competent estimator. The majority of estimators have their own individual ideas as to the manner in which the work is taken off, the form in which notes are made and preserved in the working up of the final estimate; generally, it will be found that such notes, even when preserved in an estimating book are so obscure that it is almost impossible for any other man to check the results.

The checking of the cost estimate is, as mentioned before, much more certain; mistakes can be discovered for the reason that all branches of work have certain unit prices, varying between usual limits. Any seeming inconsistency is at once apparent to the eye and can be investigated and either confirmed or revised.

We must, therefore, establish some system of recording the quantities as they are taken off, part by part, in order that we may have the opportunity of independently checking and preserving the notes in complete detail. This is preferably done by such a blank form as is shown in Fig. 13. If a form of this kind is used we secure all these advantages, and in addition we have on file a complete statement of the job so itemized and in such form that, should the job be secured, changes or corrections in any of the working plans over the preliminary plans on which the work is based can be estimated and noted. In cases where preliminary plans are returned—with possibly, a delay before awarding the contract—and then working plans are furnished, it is well to check off at once on the "Itemized Quantity Estimate" the quantities shown by the working plan. Many times, the preliminary plans are not made complete in all details, with the result that when the working plans are prepared and all details worked out, revisions and changes will be necessary in the general plans. If, as is usual, the preliminary plans have been returned, the only record remaining in the hands of the contractor is this itemized quantity estimate.

Identification of Plans. It may be well, at this point, to lay all the stress possible upon the fact that any contractor receiving a set of plans and specifications should at once mark on each sheet the date and his name in ink (rubber stamp preferred), so that in case the work should be awarded to him he may call for the exact set of plans and specifications which were in his possession at the time of making his bid. This is not advocated because we must be suspicious of dishonesty or manipulation; but it is a fact that in the preparation of various sets of plans and specifications there is always the opportunity for typographical errors, failure to bind in all the sheets, omission of details, and neglect to make revisions and corrections on some of the sets. Besides, when many revisions have been made (and especially where the letting of the work is delayed from time to time, because of such revisions and the receipt

of additional proposals) there is accumulated a multiplicity of blue prints, etc., which are out of date. If these sheets are preserved they are a menace to the contractor, as they may, through error, not be replaced by the new sheets. On the other hand, if they are destroyed there is nothing with which to compare the revision, as the architect or engineer in nearly all work makes his revisions by erasures and additions to the original tracings, and, therefore, the tracings themselves bear no adequate means of identifying the original drawing.

These same conditions apply during the entire construction of the work, and generally the first plans are called in and the new plans substituted. The writer knows of one case on a large and important piece of work where the engineering company made a rule that no revised plan would be issued until the first plan had been returned by the contractor and destroyed, and from this rule they permitted no deviation. It was found that the first set of plans was deficient in many details and as a result the quantity of work was largely increased. The method adopted by this engineer, whether prompted by proper motives or not, had the effect of covering up the mistakes and discrepancies in his original plans. As the work was complicated and constantly changing in size of members, etc., the contractor was placed in the position of unwittingly doing more work than he had really contracted to do..

The "Itemized Quantity Estimate", as proposed in Fig. 13, is the result of a personal experience in errors in quantity estimating and in losses due to just such conditions as have been mentioned. It is for the purpose of providing, so far as possible, against future errors that this form has been developed. The form has been used, and its use has demonstrated that with proper care a reasonable amount of certainty may be expected. The quantity estimate, when prepared in the manner suggested, is also of great value when the time comes to submit a schedule of quantities and prices for the preparation of the monthly estimates, and it naturally forms the basis for each month's estimate of the amount of each class of work. By this method the preparation of estimates is so simplified and systematized that the estimates will form a definite series, so tied together that at all times they show exactly what amount of work and what percentage of the work have been accomplished.

Analysis of Quantity=Estimate Forms. It will be noted that the "Itemized Quantity Estimate" bears for all practical purposes the same style of heading as that previously given for "Outline of Work", Figs. 5 and 6. By carrying through this general heading and the same estimate number, all of the estimates, plans, and papers are preserved in their proper places.

Item Column. It is expected in using this blank form that each calculation forming a part of the determination of quantities shall be set forth under each item of work. As there generally will be several calculations for each kind of work, the separate items will show very prominently in the first column with little chance of any being overlooked.

Location Column. Column 3 is important. The location of that portion of each kind of work under process of calculation should certainly be noted. If this is done, how much easier it is to check the quantities at any time, and how much more definite are the estimate and the connection of the quantity estimate with the proposed work.

Columns 4 and 5, bearing as they do a reference to plans and specifications, furnish a very ready index with practically no extra labor to the estimator.

Dimension Column. The next division, including Columns 6 to 9, furnishes all the necessary dimensions for determining the kind of work entering into the job when used in connection with the unit columns for the various classes entering into the item "Principal Work". It might be well to bring out here the point that the classification of work as called for by plans and specifications is much simpler than the classifications of different items of work which must be estimated by the contractor, and yet he must retain the specified classifications, either for unit-price bidding or for submission of schedules in lump-sum bidding or percentage contracts.

Unit and Total Quality Columns. At the right-hand side of Fig. 13, are arranged the unit and total quantity divisions required by the contractor. The unit column is intended for the usual multiplier or divisor in order to place within the total column the quantity as based upon the dimensions. The headings for these columns are to be written opposite each main item of work, according to the practice of the individual contractor.

Referring to Fig. 13, we see various illustrations of the manner in which the main items of work may show their location, part by part; the dimensions of such portions, immediately followed by the secondary classifications into which the main items must be divided in order that an intelligent estimate may be made. In presenting these examples no attempt has been made to illustrate any particular kind of job; instead, various sorts of work are given which in all probability would occur in any kind of construction.

The use of this blank can be made universal, but the individual application must depend upon the contractor's line of work and his own ideas as to its separation into component parts, and the latter, in turn, naturally depend upon the manner in which he keeps his cost record. The advantage of the form lies in the fact that, instead of taking off different parts of each main item separately and repeating over and over again the given dimensions, the main item and dimensions are given only once and the other quantities are arrived at by the use of their proper units.

Discussion of Special Items. Excavation. Let us refer again to Fig. 13, and consider the first item, "Excavation". In any piece of work it will be necessary to give several dimensions in order to cover the shape of the excavation and varying depths. The quantities should be, of course, in cubic yards; therefore, the unit column carries the divisor 27. In excavation, however, it is generally the case that shoring or sheeting of the banks is necessary, and the amount of such work is directly dependent upon the same dimensions, viz, the perimeter of the excavation times the height; therefore, the unit column under this heading states in inches the average thickness of all timber required, the total quantity being given in thousands of feet, board measure. In case the excavation is of such a character that gravel may be expected, this may be used in the construction and the depth at which it can be obtained, having been determined by test digging, we find in the unit column "$\frac{1}{2}$", or any other proportion. Inasmuch as in any large job several detailed calculations such as this are necessary in order to arrive at the grand total, we have the divisions of each item grouped so that each kind of work is stated exactly in proper relation to the work as a whole. If any item shows no such divisions investigation can be made to ascertain whether this was overlooked. Such a method

also allows a more intelligent pricing of the work, as the complicated portions are automatically emphasized and may be priced accordingly, whereas, by any other method it is more than probable that averages would be taken over the entire work.

Concrete. The same conditions apply to even a greater extent to "Concrete", the second item of Fig. 13. Concrete as a main heading will, in general, be divided into the main items of "work where the proportions of the concrete mixture are varied", "work where any one mixture is to be used". We then have the unit item under concrete containing a divisor 27, since we wish our concrete quantity in cubic yards. The number of forms required is directly dependent upon the square feet of surface area given by the main dimensions. In the unit column is placed a multiplier giving in inches the thickness of the lumber necessary to form the facing, studding, braces, etc., all of which can be quickly calculated. After some experience, this factor becomes a standard for various classes of work, giving us quickly our quantities of forms in thousands of feet, board measure. The method applies to centers as well as to forms, the resulting quantities being in thousands of feet, board measure; a separate item is required to list the amount of piling necessary in case of false work for bridge construction.

Reinforcing Steel. In building construction the reinforcing steel for various designs can be obtained from tables on a basis of unit pounds per square foot of floor and lineal foot of beams and columns, and these units should be placed in the unit column opposite the totals listed by dimensions and by pieces.

If there is more than one kind of steel, more than one column must necessarily be used for itemizing on account of the difference in prices. If the structure is entirely of steel or is a large reinforced-concrete job with complications in the sizes of bars and kinds of reinforcing, or in standard structural shapes, then steel should be made a general item of work. These unit columns then become the proper method of separating steel bars and shapes into their proper classification for standard prices and standard plus extras. In a complicated case with many divisions which requires more columns than are provided by the standard sheet, additional columns may be cut from other sheets and pasted on the right-hand side. By folding in these columns, the size of the sheet will not be changed.

Waterproofing. Similar extensions for concrete work are at once apparent. For instance, waterproofing, if incorporated in the mass, is an element of the cubic yardage and should be listed opposite that portion of the work where waterproofing is required. If waterproofing is of the sheet or surface application type, its quantity naturally depends upon the square feet of surface covered, which is shown by the dimensions of that part of the work. The unit column for waterproofing would, therefore, show unit pounds of compound per cubic yard, or thickness in fractions of a foot to give results in cubic feet.

Cement Finish, Floors, etc. The same conditions apply to cement finish, filling, wood floors, marble floors, painting, plastering, etc., as all these items will appear opposite the portion of work where they are to be used. As the dimensions of such parts of the work are clearly stated, it is only necessary to place in the proper unit column unit multipliers or divisors to secure the results on the desired quantity basis. This unit is naturally preserved along with the other complete data, and if, for instance, it is desired to ascertain the additional cost when the cement finish in certain portions of the work is changed from $\frac{1}{2}$ inch in thickness to $\frac{3}{4}$ inch, the information is obtainable almost instantly; if this method has not been followed, it will be necessary to refigure the plans.

Timber and Lumber. Timber and lumber can be taken off in the same manner, using the columns for the different sizes of timber at the various prices, and using the unit column for unit feet, board measure. If the structure is a large one and many different-sized pieces are called for, all that is necessary is to provide additional columns, as explained above, under steel.

Brickwork. Brick work is peculiarly adapted to this method of estimating. The various portions of the work are stated in the "location" and "dimensions given" column; we then use our secondary columns to obtain the quantities of common brick, face brick, special brick, and in each case the separation is made directly opposite the respective locations, but the total is reached just as accurately. The opportunity is also afforded for the statement, in parallel columns, of the amount of brick by "wall measure" and "kiln count".

Remember that this should be an axiom of the engineer or

contractor, that *any estimate must be made by detailed calculations.* If such detailed calculations are recorded as provided for by a standard form, the totals are readily obtained; but a lump-sum quantity estimate obtained from scratch-pad calculations can never be separated without recalculation and uncertainty.

Check on Totals. The extreme left-hand column "Group Items with Brackets" makes doubly sure the obtaining of correct totals and adds materially to the clearness of the estimate. If, for instance, we have in the "Item of Work" column the heading "Concrete" and then follow on down the sheet with numerous items filling out the right-hand columns, then by the addition of a bracket inclosing these items down to the total, the eye is enabled at a glance to read the results without confusion.

The illustrations given are sufficient to show the wide application of the method suggested. The advantage of this form of detail quantity sheet is not limited to the estimate, for, in case the contract is secured, the sheet is relied upon for the ordering of material and the order in which it is to be delivered—all because each class of work is given by its location, by the unit applying to such work, and by the quantity required for each portion. In any lump-sum estimate this information could not be obtained without recalculation.

The quantity sheet also works directly with the methods of cost analysis and the office and field system connected therewith. Above all it is of importance, during the period of construction and analysis, in the division by sections and the recognition of them.

In taking off the quantities from the plans, use common white crayon to check each item and each portion of the work as the quantity estimate is tabulated. When the estimate is completed and the plans are to be returned to the architect or engineer, or forwarded to a competitor, wash the prints with a soft rag soaked in gasoline and all defacement will be removed.

Calculating Machines. In the preparation of the itemized quantity estimate where many extensions are required, some form of a non-listing calculating machine is almost indispensable. Such a machine insures accuracy and saves time and extra help, and, because of its speed, an estimate can be prepared, checked during preparation, and delivered in a fraction of the time required by

ordinary methods. The cost of such a machine is not excessive and it is well adapted to all forms of work in a small office. In the larger office there should be in addition a listing adding machine, in order to check the totals.

Slide Rule. The slide rule should not be used in estimating quantities or in multiplications and extensions. This is not said with the idea of underrating the slide rule; but in cost analysis the latter serves a far better purpose in connection with the engineer's practice, and for this purpose it is indispensable if time is any object.

ITEMIZED COST ESTIMATE

Comparison with Previous Forms. Passing to the "Itemized Cost Estimate", which is directly dependent on the "Itemized Quantity Estimate" and the next and final step in the estimate department, we propose the forms shown in Figs. 14 and 15. The form shown in Fig. 14 is to be used where all the work contemplated is to be performed by the contractor's own organization and where, as a result, it is necessary to enter into full detail as to the costs of each and every item of work. The form shown in Fig. 15 is to be used for the tabulation of those portions of the work where the contractor accepts sub-bids in lieu of making his own estimates, or in addition thereto.

The headings of these forms are practically the same as for the Itemized Quantity Estimate, except that the estimate is made more definite by stating "for whom" the bid is prepared and the "work included in estimate". This is good information, especially if only a portion of the work is included and figures are made for general contractors. Perhaps the same work may be the subject of two or more estimates—for example, a preliminary one to the owner or architect and a final one to them or to the general contractors—and the notation explains any discrepancies.

Analysis of Cost Estimate Forms. Referring again to Fig. 14, we see that Columns 1 and 2 are taken directly from the "Itemized Quantity Estimate" recapitulation of totals, Fig. 13. These principal items are now dissected to analyze the cost which in all cases demands the separation of material and labor, and also generally requires several items for materials which go to make up the principal item. Moreover the labor must many times be separated

The labor cost of pile driving is made up of two labor charges, viz, (1) (a) rigging and (b) removing outfit; (2) driving. Let us assume a cost to rig and remove outfit of $300, and two jobs—one of 3000 lineal feet and one of 30,000 lineal feet. When it comes to the actual driving, the cost should not vary to any great extent, yet the rigging and removing in the first case is 10 cents per lineal foot and in the second case 1 cent. Wild bids can many times be explained this way.

Be sure to analyze each item, and, following through the job in imagination, determine all the elements which enter into each operation. Build the job step by step in your own mind and thereafter you will never set down any guess costs or make snap judgment as to the cost because it agrees with the cost on another job.

As an illustration of this form of estimate, suppose we take a concrete job of any character requiring excavation, piling, foundation concrete, superstructure concrete, forms and centers, reinforcement, and cement finish. This estimate is carried to completion in two ways: (1) For lump-sum bid; and (2) for unit-price bid. General charges are included, but for the present we will pass the detail determination of them. A special discussion will be given to this element of the cost, which, in its insidious way, eats into the anticipated profits, sometimes to their final annihilation. Upon the face of the estimate sheet we have every detail calculation of the cost and why. By simply referring to the Itemized Quantity Estimate, Fig. 13, we have detail calculation of the quantities and why. There is no guesswork about such a system, for, even if some items are uncertain of determination from the plans, yet these stand by themselves and can be verified or mention may be made in the proposal that a certain amount has been estimated. Any questions by the owner or architect can be answered at once and definitely. Any eliminated items can be deducted without recalculation.

Unit-Price Bidding. On the lower half of the Itemized Cost Estimate, Fig. 14, an illustration is given of estimating a job for unit-price bidding. It may be well to go into some detail in the analysis of concrete work as the estimator must do before itemizing. The itemization given is sufficient for all practical purposes but back of this statement of quantities and units lies

in each individual job should be estimated as carefully as for any other item. It does not matter whether the cost is estimated on the basis of floor or of concrete surface, or upon a basis of feet, board measure, of lumber; the result is the same just so long as this important branch of concrete construction receives individual consideration.

(5) Cost of Work as Affected by Centers: There may be a misunderstanding as to the necessity of separating forms and centers. "Forms", properly speaking, consist of the timber work to form those members of concrete construction—columns, girders, etc.—which must possess a certain regularity of shape and evenness of outline. "Centers", properly speaking, consist of timber supports for floor slabs, bridge centers, etc. Where the great amount of labor on timber work is expended solely for the purpose of supporting the mass at a uniform or definite height the unit cost is much less than where it is expended for forming for individual members.

(6) Cost of Work as Affected by Reinforcement: Fabricated reinforcement will cost much more delivered to the job than will loose bars; but, on the other hand, it will cost less to place in position. It is to be expected that a job where the reinforcement is made up of small bars of less than $\frac{3}{4}$ inch, will cost much more per ton for labor than where the sections are larger. Good judgment must prevail in this particular. It is very seldom that an estimate will be made based on the labor and the tonnage of each size of bar. It is the better way to strike an average labor price with reference to the proportion of different sizes of reinforcement to be encountered in the work. It is well to remember, in making the estimate, however, that it takes practically as long for the steel man to place and wire a $\frac{3}{8}$ inch bar as a $\frac{3}{4}$ inch bar, whereas the tonnage is only one-quarter as great.

(7) Cost of Work as Affected by General Charge Items: There are a great many items of the cost of concrete work which are not included in the cost of material, labor, forms and centers, reinforcement, etc. These costs consist of installation of machinery, and its removal, coal, oil, repairs and fittings to machinery, general charge items such as rope, bolts, etc., and the loss on petty tools such as shovels, etc. Such costs on the basis of so much per cubic yard, are to be determined from the percentage of labor on different classes of work.

Cost-Estimate Form for Sub-Bidders. Fig. 15 contemplates that when any portion of the work is to be sublet, the contractor will yet make his own estimate of the quantity of such work and the fair unit price. This method serves as a check on the sub-bidders and also provides a basis for determining the unit prices at which the various sub-bidders are estimating.

The first three columns are used for the contractor's own check estimate, while the balance of the sheet is for tabulation of sub-bids.

Relation of Cost Estimates to Previous Forms. Both forms, Figs. 14 and 15, should measure $8\frac{1}{2} \times 14$ inches, as this size is just as convenient for filing and the extra length saves labor in forwarding totals.

The Outline of Work, Figs. 5 and 6, the Itemized Quantity Estimate, Fig. 13, and the two forms of Itemized Cost Estimate, Figs. 14 and 15, should be bound together in order, preferably on the side, in a loose-leaf binder. Each estimate as made should be placed in this binder in the order of the estimate number, and an index should be made. In a large office it is better to file the Outline of Work and Itemized Quantity Estimate in one binder in the estimating or engineering department, while the Itemized Cost Estimate is personally in the hands of the engineer, contractor, or of a strictly confidential employe.

Estimated Cost of Equipment. The amount to be added to the estimate for rental or depreciation of plant and equipment depends upon the "scheme of operation". It is suggested that in construction work, there should be added to the estimate the difference between the cost price of new equipment and its second hand sale value; good judgment must be used to determine this depreciation with regard to the character and probable wear and tear of each class. The depreciation, however, usually will vary from 40 to 60 per cent. In the case of equipment on hand, it is a matter of choice whether we add a rental in addition to profit or whether we consider the use of such a plant as included in the added profit and thus presumably gain an advantage over less fortunate competitors.

Maintenance of Equipment. The subject of "Maintenance of Equipment" is, however, a separate charge over and above any rental or depreciation and is a true cost to be charged to each job.

Various methods have been suggested, but for simplicity the writer advocates a Repair Account which forms a part of the General Charge. This subject will be treated in detail later when all of the proper charges to this account will be listed. Properly speaking, this is "Material-Cost Bookkeeping" and should be treated under Construction.

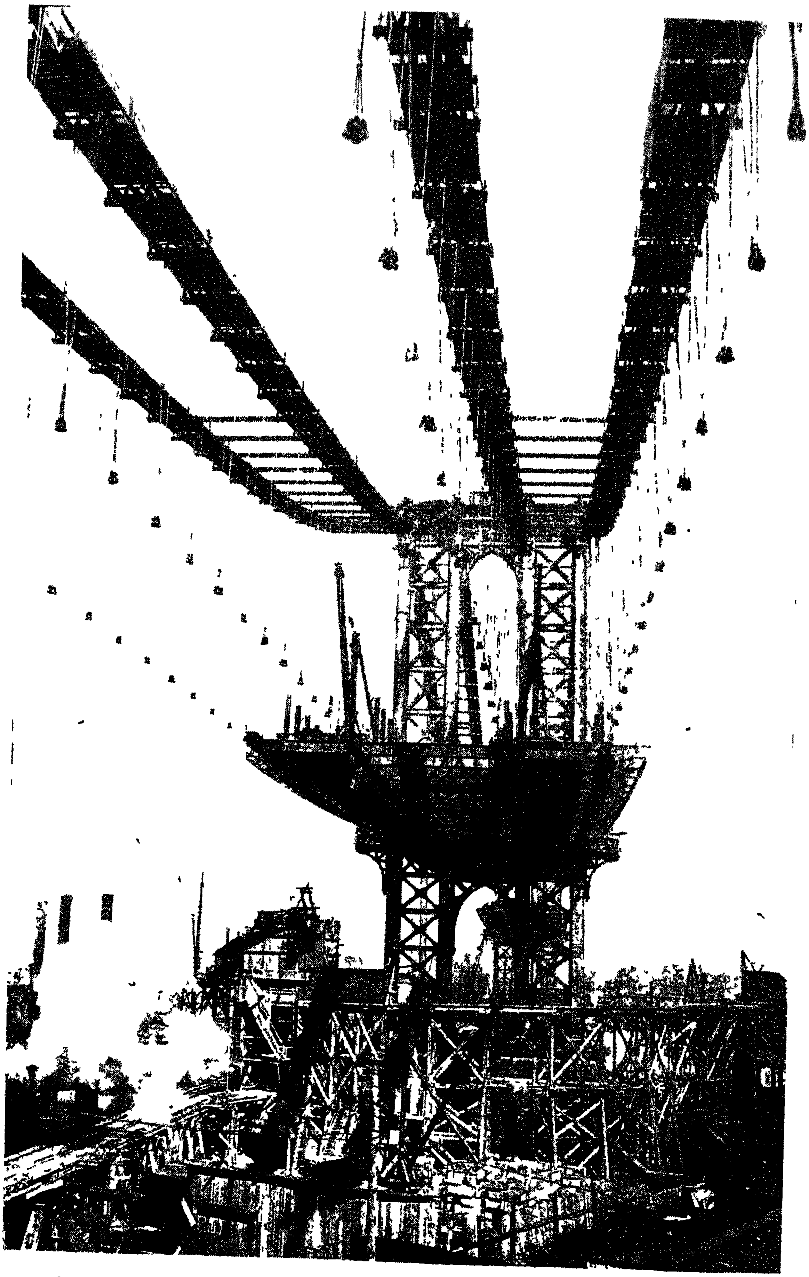
Rental or Depreciation of Equipment. Only in exceptional cases should rentals or depreciation be added as part of the unit cost of any item. If they are added to this cost, a fictitious idea is obtained of the estimated cost of the item, and moreover the total amount set aside for this cost can be obtained only by recalculation. This method is, besides, cumbersome and indefinite and will not stand investigation and analysis as will the percentage method of General Charge. The method of charging out the difference between the cost and second hand selling value in the case of new equipment is based on the theory that the repair account is charged to the job and that if the plant is properly looked after the machinery is always in good second hand condition and may be carried so on the inventories. If a rental is charged—or if equipment is leased to others—10 per cent per month, or a life of ten months on the first cost, is not unreasonable, except for rolling and heavy equipment, or on large contracts. This theory is based on the idea that the plant is a profit-making investment, and repairs and upkeep are worth the price.

Summary of Points in Making Estimate. In making the estimate, arrange the items in the logical order of sequence, as they would naturally occur in the actual construction. If the estimate is built up in this way there is little chance of overlooking items. A contractor who lacks imagination is certainly in the wrong line of work. The process of visualizing the proposition step by step will many times call attention to items which do not appear in the plans and specifications. We cannot anticipate all contingencies, but it is possible to build in the mind a model of the particular proposition.

Cost Data Subject to Wide Variation. Throughout the presentation of this cost-analysis system, the author has consistently refrained from stating any cost data based on actual experience, believing that such costs are of little value, except to the individual with whom they occur, and that they apply to his locality only.

The purpose has been to emphasize the methods whereby each individual in any locality can analyze and develop his own system.

Material costs are readily obtainable. Labor costs are uncertain, depending as they do on character, class, nationality, and environment of men employed, and the circumstances and difficulties surrounding the work. We are writing for the man of experience who will accept suggestions as to methods, and for the young man who desires the groundwork upon which to build up an organization. The contractor knows his labor and what he can expect as a day's work. The young man should serve his time with an established organization, keeping his own notes and forming conclusions which, if supplemented by system and proper consideration for overhead and general-charge expenses, should enable him to start well prepared for prosperous experience. The young man can gain ideas of the efficiency of men by service as a timekeeper or sub-foreman, but will, in the majority of cases, miss many items of cost which do not appear in the field and which are confidential with his employer. It is to supply a lack of information on this subject that this paper will include actual results on costs other than material and labor.



MANHATTAN SUSPENSION BRIDGE, NEW YORK CITY, IN COURSE OF CONSTRUCTION

Photo by Brown Brothers, New York City

COST-ANALYSIS ENGINEERING

PART II

A COST-ANALYSIS SYSTEM (Continued)

GENERAL-CHARGE OR EXPENSE ITEMS

Importance of General-Expense Items. The items which go to make up the General-Charge division of cost may be designated by various titles and may be proportioned to the estimate in different ways, but, regardless of their divisions in the schedule, their presence in connection with any work is as sure as the forces of nature. They are just as sure and certain a cost as material and labor and are really more destructive to profits, because of their insidious accumulation through gradual and apparently trivial daily expenditures. For our personal convenience we choose to apportion these items to labor, although this is proposed not as a rule but as a suggestion. They must be considered in the estimate, and yet, to a large extent, they cannot be determined except as the result of actual experience.

The author well remembers that when he first began estimating in construction work it was a matter of surprise and bewilderment to him why jobs, which in cost of material and labor fully realized the estimate, still proved a loss—or nearly so. It was a considerable time before it was recognized that the many small purchases were, in the end, the destroyers of profits, and that it was not safe to pass them by with the expectation that they would not be important enough to justify consideration in estimating. Some do not or cannot understand this but look on these items as contingent expenses unnecessary to include in the estimate. This is very far from the truth, for such expenses are as truly cost items as are material and labor.

Methods of Application. Some engineers and contractors advocate an allowance per unit of work, as per yard of excavation, concrete, etc., to take care of these charges. Others add an amount to the estimate, sufficient in their judgment to cover the total item.

Errors in Unit-Allowance Method. The first method is altogether wrong for the following reasons:

(1) The addition of any such amount to a price per unit of work is deceptive in its after-effects.

(2) Such a method can never arrive at anything but a "guess".

(3) The estimator is yet unborn who can differentiate the cost per unit for general-charge items on several classes of work and secure an intelligent result.

Suppose we imagine an excavation for a pier foundation for a bridge requiring excavation, pile driving, and concrete. To do this work requires pumping, and a great deal of it if the pier is located out in the stream. Invoices accumulate at the office for coal and oil, and the actual coal and oil is delivered to the job and used as occasion demands. Excavating, pile driving, and concreting proceed practically simultaneously. Under these circumstances, how can we arrive at any fair cost per unit for fuel and oil? If the attempt is made to distribute the cost in any proportion to the various kinds of work it can only be done by arbitrary methods, which will not bear analysis.

Take another item—for example, Petty Tools. In this classification belong shovels. What contractor is wise enough or sufficiently in touch with his work to say what proportion of this cost should go to excavation and what to concrete?

Take the Fittings and Repairs account. Who can say with any certainty how much of the repairs to pumping machinery should be charged to excavation and how much to pile driving or to concrete?

The fact is that all these charges are directly dependent upon the kind of work under construction, and the various jobs will show a greater or less cost along this line in direct relation to the difficulties encountered.

Distribution by Percentage Method. Therefore it follows that the various kinds of work should be separated, according to each contractor's experience, into definite classifications, with respect to the difficulties surrounding construction; from these the percentages to be added to the estimated cost should be ascertained to take care of all the items of general charge. Such classifications have been prepared by the author for his own work, and it has

been found really surprising how closely the percentages of cost of these items for work of similar character will compare, even for both small and large jobs.

Percentage on Labor Cost Only. The consideration of the method of application of such percentages is important. If the percentage is added to the complete estimated cost of material and labor the results will show wide variation, due to the differences in cost of material. If proportioned over material only, the result will vary still more. The only logical method is to distribute this cost by a percentage on the cost of labor only. This theory is based on the following facts:

(1) The labor for the same kinds of work is more nearly constant than the cost of material.

(2) The contractor must always furnish labor, whereas material may be entirely eliminated; it may be reduced in price by concessions as to transportation (as in railroad jobs with free haul); or, as in cases of work at some distance from railroad facilities, it may be greatly increased in price by delivery charges.

(3) The higher the grade of work the more the cost of labor and the more the cost per man-hour.

(4) The more difficult the work the greater the cost of labor and the less the output per man-hour.

(5) The higher the grade or the more difficult the work the less the output per dollar of expenditure for payroll.

Labor Cost, the Measure of Efficiency of Management. Granting the foregoing conditions, the cost of labor is the measure of efficiency of the work management. To secure results which are comparable and thus to obtain correct ideas of costs as a basis of efficiency, the general charges must be determined by a percentage on labor, this conclusion being always governed by a due consideration for the kind of work. The more thought given to this conclusion the more it will impress itself on the individual as a truth. Consider any type of construction with which you are familiar, and the method will gain in significance with each illustration considered.

Take a case of heavy wet excavation. The cost of labor is naturally more than for dry excavation, and so is the cost of all the items going to make up the general charges. For this class

of work the increase in general charges will show most in Fuel and Oil, Fittings and Repairs, and Expense. High-grade labor will not be employed on this class of work. But suppose we have stone masons building a piece of masonry which must be carefully cut and fitted. Fuel and Oil, and Fittings and Repairs are low in this case, but Expense will be large in proportion to payroll on account of dressing of tools, etc., and the output per dollar of labor is small. The two cases are not comparable and belong in separate classifications, but they are mentioned as illustrations of the principles involved.

This method has been applied to 125 jobs covering ten years of work and it has been found to give definite comparable results, whereas other methods make the estimates largely a matter of guess. In using the percentage method it is always wise to prepare a check estimate by a mental review of the job under consideration and deciding, in accordance with past experience, whether or not the amount determined by percentage seems reasonable.

Discussion of Divisions of General Charges. The schedule, or division suggested for General Charges, is as follows:

(1) Contract Bonds, Maintenance Bonds, etc

(2) Liability Insurance—comprising Employers' Liability, Public Liability, and Workmen's Collective Insurance.

(3) General Expense—items as per list.

(4) Petty Tools—as per list.

(5) Fuel and Oil—as per list.

(6) Fittings and Repairs—as per list.

(7) Commissary—where necessary, as per list.

(8) Interest.

(9) Machinery and Equipment.

(10) General Office, and Supervision.

In establishing any such schedule as the foregoing, it will be recognized that, unless a general rule be adopted as to what shall be charged under each heading, the bookkeeper cannot, day by day, record the general charges in such a way that all jobs, past, present, and future, may be compared with reference to the same item. Inasmuch as bookkeepers change while the system must continue, it is well to preserve a decision once made by preparing a stand-

ard list of the ordinary purchases. Such a list will be shown for each of the headings given above.

In considering Bonds and Insurance, information and rates are given at considerable length. A man located in a large community may not find this data as valuable as the man who is not so closely in touch with a well-informed agent, but, for reference and comparison and for purposes of estimating, this information is indispensable. However, it is advised that wherever possible an agent of the bonding and insurance companies be called upon to verify the information. Incidentally it should be noted that liability insurance is computed on the payroll at various rates, according to the hazard of the occupation, and this change of rate substantiates our contention that all the items of general charge should be adjusted to the job. Since we have insisted that, in estimating, material and labor should be separated, there is no difficulty in the application of this method.

It will be found that for different kinds of work under various conditions, the proportion of cost of labor with respect to the cost of material will run pretty uniformly, and if a record is made of these percentages it will assist a great many times in approximate or preliminary estimates.

BONDS

Classification. The information herewith furnished, with reference to bonds, is in accordance with a manual of fidelity and surety bonds isssued by a well-known bonding company *for past work*. Contract bonds are classified as (a) construction; (b) supply; and (c) maintenance.

Construction and Supply Bonds. Construction *contracts* are those in connection with work which forms a part of, or becomes attached to, real property when the contract in question is completed. When the contract for the repair or erection of any work classified as construction includes, within the same contract bond, the furnishing of any building material, the entire contract is rated as a *construction bond.* When the contract is solely for the furnishing of building material and does not include any installation, the contract may be classified as a *supply bond.*

Premiums. Construction Bonds. The premium applicable to construction bonds for usual kinds of contract work is based on

a rate of one-half of one per cent per annum on the contract price. Note that this rate is figured on the contract price and not on the amount of the bond; also note that the premium is per annum, so that, should the work extend over more than one year, allowance must be made for renewal of the same premium. It should also be understood that the premiums are due from the date of the bond, whether actual work begins immediately or not.

Supply Contracts. Premiums for supply contracts may be figured at the rate of one-fourth of one per cent per annum on the contract price, and include the furnishing of supplies, material and machinery of stock design. Machinery, etc., made to special order requires a premium of one-half of one per cent, the same as a construction contract. Any labor connected with the installation of supply contracts must be figured at the construction-contract rates.

Proposal Bonds. Many times it is necessary to furnish a *bid or proposal bond.* The usual rate for a bond of this character is $5.00 each, and this premium is considered fully earned whether the proposal is successful or not. Should the proposal be successful the premium for this bid bond may be applied as a part payment on the premium for the contract bond, provided the bond is taken from the same company.

Construction Contracts. The minimum premiums on construction-contract bonds are as follows: (1) where the bond does not exceed $500 and the contract does not exceed $1000, bonds $5 each; and (2) where the bond is over $500 or the contract is over $1000, bonds $10 each. It will be noted that these minimums require that a contract shall equal $2000 or more before the regular rate of one-half of one per cent would equal the minimum premium.

Maintenance Bonds. Maintenance bonds are of two kinds: viz., those covering the guaranty of the entire construction work; and those covering the guaranty of only a part of the construction work. In the first case the premium rate for maintenance bonds is $1.25 per thousand per annum computed on the contract price, and in the second case $1.25 per thousand per annum computed on the value of the guaranteed work. The premium for the maintenance bond for the entire term of the guaranty is due in advance at the time the construction bond is issued.

Establishing Credit with Bonding Company. It is just as important to the contractor to have a credit line with some well-established bond company as to have a credit with his bank. The standing of any contractor with the bonding company depends on two conditions: financial standing, and moral hazard. Within reasonable limits a bonding company will extend greater favor to the good moral hazard than to the contractor of better financial standing but with a poor record for fulfilling his obligations.

Many states and municipalities, and also the United States government, have special requirements regarding bonds, and it is always well to consult an agent to be sure that the standard rates

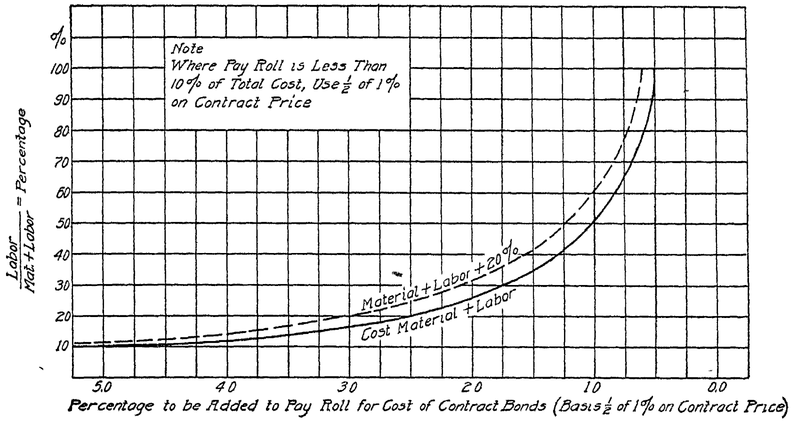


Fig 16 Curve Showing Percentage to be Added to Payroll to Cover Cost of Contract Bonds

apply on any new work. The foregoing rates are standard for ordinary work, old manual

Method of Calculating Bond Cost. Inasmuch as the premium on bonds, regardless of the amount of the bond, is computed on the total amount of the contract—which includes material, labor, general-charge items, and profit—we must either compute the bond cost separately from other items of general charge or increase the rates in proportion to the percentage which the payroll bears to the cost of material and labor. The more accurate method is to complete the estimate, except for the bond, and then to add the cost of the bond plus the same percentage of profit as is used for the general estimate.

For all practical purposes, however, sufficiently accurate results are obtained by including the bond cost along with other items of general charge. As our system requires that material and labor be separated in the estimate, we are able to arrive at once at the percentage the payroll bears to material and labor. By referring then to chart, Fig. 16, the proper per cent to be included in the general charge and added to the payroll may be ascertained.

For instance, if the labor represents 50 per cent of the cost of material and labor, the percentage to be included in the general charge and added to the payroll is 1 per cent. If the labor represents 20 per cent, or one-fifth of the cost of material and labor, the percentage to be included in the general charge and added to the payroll is $2\frac{1}{2}$ per cent. Naturally, if the contract is all labor and therefore all payroll, representing 100 per cent of the cost, the rate will be one-half of one per cent. At 10 per cent labor the per cent to be added to the payroll increases to 5 per cent, but a contract with such a small amount of labor is unusual, and it is recommended that jobs of this character with 10 per cent or less labor be calculated by the method first suggested. When we have a job with 10 per cent or less labor we are verging on a supply contract, and it is probable that the cost of the bond can be decreased by a proper division of the work under the headings of construction and supply.

LIABILITY INSURANCE

Features of Liability Insurance. Liability insurance is not accident insurance, as some employes seem to understand. Contractors' liability insurance (briefly known as E. L. policies) is essentially insurance against the contractor's liability for damages because of accidents to any of his employes It is, in fact, a contract between the contractor and the liability company whereby, in consideration of a certain fee to be paid by the contractor, the liability company agrees to furnish the first or emergency medical aid and to save the contractor from any and all claims for damages up to the limits of the policy. The usual limits of the policy are \$5,000 and \$10,000, meaning thereby \$5,000 for any one individual judgment or \$10,000 for any one accident—the latter being the limit for an accident when a number are insured and where the

aggregate amount of judgments does not exceed the amount named. Almost any limits will be accepted by the liability-insurance companies but the ones given are usual. Any increase of the limits is naturally accompanied by an increase in the rate of premium.

The contractor should consider that his rating or credit with his liability-insurance company is almost as important as his credit with his bank or bonding company. Both the bank and the bonding company gage the contractor by his financial standing and by his general reputation for fulfilling and satisfying his obligations. The liability-insurance company judges the risk in light of its past experience on similar work, but tempers its decision by the standing of the contractor and what is known of his experience, methods, and disposition to assist the company to secure reports, records, and settlements. A contractor, by establishing a good reputation may gain the advantage of a close rate, while one with a poor record may even be refused by one company after another. The interests of the contractor and the liability company are identical and there should be close co-operation.

Liability Premiums. The minimum premium for liability insurance is $50, and the only way to avoid an increase in cost on this account, when many small contracts are handled, is to secure a general, annual, or blanket policy covering all work of any character. Naturally on large work this feature is of no consequence.

Public-Liability Insurance. The possibility of accidents to the public and to the employes of others must be considered, especially when the work is located in a city or town where others are working on the same contract. To provide for this contingency public liability insurance (P. L. policies) is furnished at a somewhat less rate than is employe insurance.

Workmen's Collective Insurance. Workmen's collective insurance is now required in some states and is a feature of the insurance cost which will require more consideration in the future. Briefly, this class of insurance may be considered as accident insurance. The rates vary with the class of work. The benefits are:

(1) In the event of death within 90 days, a sum equal to, but not exceeding 1 year's wages, sometimes limited to $1500.

(2) For the loss of two limbs or two eyes a sum equal to the amount payable under the policy at death.

(3) For the loss of one limb, a sum equal to one-third the amount payable under the policy at death.

(4) For the loss of one eye a sum equal to one-eighth the amount payable under the policy at death.

(5) In the event of temporary total disability, a sum equal to, but not exceeding, one-half the weekly wages for a period not exceeding 26 weeks, such sum not to exceed $750 in respect of any one person injured during the policy year.

These benefits are for accidents occurring only during working hours, but by an additional charge of 15 per cent the policy may be written to cover the whole 24 hours.

Discussion of Rates of Insurance. The brief abstract of standard rates, Table I, gives a general idea of the cost of employers'-liability, public-liability, and workmen's collective insurance. This table is not intended to cover the entire field of contracting work, and has been prepared from a manual of recent but not current date. The examples are given to serve only as a guide in general construction work.

The rates in the table do not include the hazard due to blasting, unless specifically mentioned. The rates given are for Illinois, Iowa, Missouri, and Nebraska, and for all other states by use of the proper differentials.

How Cost of Liability Insurance is Borne. Many states, by legislative enactment, have very materially increased the cost of liability insurance. The intention has been to protect the interests of the workman, but it must be remembered that the public pays the bill in the increased cost of all building operations. The workman is indeed deserving of a certain protection in his work, and the economic interest of the community at large justifies a recompense to him or his family for injuries or loss of life, but this result cannot be obtained without a corresponding increase in cost of every building project.

In other words, every contract carries, in addition to the cost of material, labor, and supplies, the possible cost of human effort expressed in loss of time and suffering due to injury and death. This cost is just as true a one as are the others, is more far-reaching, and should receive its proper recognition; but only by legislative action can this cost be distributed over the entire community and

be included in all proposals—a very essential factor in estimates and proposals.

The only regret is that such laws are abused by malicious and scheming employes, abetted and assisted by attorneys, to the end that the very purposes of the legislation are defeated and the matter of claims made the subject of distrust and antagonism. Any prudent or responsible employer of labor cannot afford to be without the protection of liability insurance, for to carry his own risk invites litigation and loss. A minor may have an accident today but can defer suit for years; therefore liability insurance should be carried in a well-established company of financial strength. Fire losses cannot exceed the value of your property, but liability for personal injuries cannot be estimated.

TABLE I

Abstract of Standard Liability-Insurance Rates

These rates should not be used for estimate purposes. Secure current rates from a liability company.

	Employers' Liability	Public Liability	Workmen's Collective
Asphalt layers—street or sidewalk, including yards and shops	$1 00	$1 00	$2 00
Blasting (add 20 per cent to E L, P L, and Coll. rates, to cover blasting hazards, except in those classifications where blasting is specifically included			. .
Bridge building, metal	7 50	2 00	4 00
Bridge building, wood	4 00	1 50	4 00
Building raising, shoring building, removing walls, foundations, columns and piers and rebuilding them	7 50	3 00	4 00
Building movers	5 00	3 00	4 00
Caisson work for foundations of subaqueous operations—with or without blasting including work under air pressure	10 00	50	4 00
Canal excavating—not for irrigation ditches	5 00	1 00	4 00
Carpenters—construction away from shop, not bridge building nor grain-elevator construction work	2 25	.75	2 50
Carpenters—grain-elevator erection	4 00	75	2 50
Carpenter work—general	1 75	.50	2 00
Cellar excavation—no caisson or subaqueous work; including digging holes and filling them with concrete for foundations for buildings	3 50	1 00	2 50
Chimney work—concrete, stone or brick construction, not structural iron or steel	7 50	1 50	2 75
Clay digging—no canal, sewer, or cellar excavating or mining	2 25	50	2 50
Cleaning and renovating stone fronts of buildings—no construction work	3.50	1 50	2 00

TABLE I—Continued

	Employers' Liability	Public Liability	Workmen's Collective
Concrete work—general	\$3 00	\$0 50	\$2.00
Concrete work—special.			
(a) Bridge abutments not more than 20 feet in height from the base including the making, setting up, and taking down of all frames and false work	3 00	1 00	2 00
(b) Bridge abutments over 20 feet in height from the base, including the making, setting up, and taking down of frames and false work	4 00	1 50	2 50
(c) Bridge or building foundations, except caisson work	3 00	1 00	2.00
(d) Bridges, including making, setting up and taking down of frames and false work, except caisson work	5 00	1 50	2 50
(e) Buildings, not grain elevators, including making, setting up and taking down of frames and false work	4 00	1 50	2 50
(f) Floors or pavements not self-bearing	1 25	1 00	2 00
(g) Grain elevators, including making, setting up, and taking down of frames and false work	5 00	1 00	2 50
(h) Sidewalk laying, including shop and yard work	1 00	1 00	2 00
(i) Unit-system construction of concrete columns, beams, roofs, walls, and floors in sections, including subsequent erection of same	7 50	3 00	4 00
Ditch digging—irrigation or drainage only; no sewer or canal building	1 50	25	1 50
Dredging by floating dredges	2 25	.50	2 50
Fences—wood, stone, metal, or concrete, not over 6 feet high—construction of	1.50	.50	1.25
Galvanized iron and sheet iron work—erecting and repairing	3.00	1.00	4 00
Gas works—laying of mains and connections; no tunneling (Employers' liability rate excludes gas explosion, inhalation, or asphyxiation)	3.00	1.50	2 50
Grade-crossing work—involving all work incidental thereto, excluding iron and steel erection work or the laying of new sewers and excluding accidents to railway passengers and by any trains except work trains	4.00	1.00	2 50
Grain elevators—concrete, including making, setting up and taking down of frames and false work	5 00	1 00	2 50
Iron men—erecting steel- and iron-frame structures, no bridge building	7 50	3 00	4 00
Jobbing work on private residences not exceeding 3 stories and basement in height, and on private stables and garages	2 00	.50	2.00

TABLE I—Continued

	Employers' Liability	Public Liability	Workmen's Collective
Jobbing work on buildings other than private residences—excluding remodeling and new construction work	$2 75	$0 75	$2 50
Marble and stone setting—inside construction only, including decoration in place	1 00	25	2 50
Marble and stone setting—outside of buildings, away from shop	3 50	1 00	2 50
Masonry work—in connection with concrete work	2 50	50	2 00
Masonry work, not otherwise classified	3 50	1 00	2 50
Mosaic work—floors only within buildings	75	25	1 00
Paperhanging	1 00	25	2 00
Parquet-floor laying	75	25	1 00
Pile driving—including timber wharf building thereon, if any	4 00	75	3 00
Pile driving for building foundations	4 00	1 50	3 00
Plastering	1 00	25	2 00
Plumbing-work—shop and outside, no division of payroll allowed	1 00	25	1 50
Road or street making	1 50	1 00	2 50
Roofers, gravel and composition only	2 00	1 00	2 00
Roofers, all other kinds	3 00	1 00	2 75
Sand and gravel diggers—no canal, sewer or cellar excavating or grading	2 25	50	2 50
Sand excavation by means of suction dredges—including loading and unloading at docks, wharves and elsewhere	2 25	.50	2 00
Sewer building—no limit of depth; with or without blasting	7 50	3 00	3 00
Sewer building—maximum depth of excavation 12 feet at any point; with or without blasting	6 00	3 00	3 00
Sewer building—maximum depth of excavation 7 feet; with or without blasting	4 00	3 00	2 50
Steam-, electric-, street-, or interurban-railway construction—earth work only, no explosives and no tunneling	2 25	(a)	2 50
Telegraph or telephone—construction exclusively	3.00	2 00	2 50
Trestle building, wood	4 00	1 00	2 50
Water towers and standpipes, erection of	7 50	1 50	4 00
Water works—construction of pumping station, and the making of dams and reservoirs; excluding erection of water towers and standpipes	3 00	50	4 00
Water works—laying mains, connections	3.00	1.50	2 50

Fire Insurance. Fire insurance is not of great importance to cost analysis in construction work, but, as it is often required, it is well to know that the rate is one-fourth to one-half of one per cent per annum short rate—that is, this rate will be proportioned to the actual time of construction. Many times arrangements can be made to join with the owner in a regular standard 3-year term policy whereby the insurance during construction may be carried at much less expense.

TABLE II

General-Expense Items Including Supplies and Repairs to Small Tools*

Belt laces	Paint (all kinds)
Boiled oil	Paint pots
Bolts	Paraffin
Boiler paint	Pitch
Brushes	Printing
Brooms	Powder
Building paper	Putty
Buildings (temporary)	Rope
Burlap	Rents (temporary)
Butts	Rivets
Cable clamps	Roofing material
Caps and fuse	Rollers
Chalk	Rosin
Clips (rope)	Sand paper
Drift bolts	Screws
Dynamite	Snaths
Emery cloth	Spikes
Funnels	Sprinklers
Handles (tools)	Steel brushes
Hasps	Steel shapes
Hinges	Strainers
Hooks	Tapes
Iron (blacksmiths)	Tar
Keys	Telephones
Lag screws	Tin
Lantern globes	Track bolts
Level glasses	Track spikes
Livery hire	Turpentine
Locks	Waste
Lumber	Washers
Molds	Wheels
Metal (Babbitt)	White lead
Nails	Wicks
Nozzles	Wire
Nuts	Wire cloth
Oakum	Etc
Office supplies	

GENERAL-EXPENSE ITEMS

Classification of General-Expense Items. General-expense items are many and various. Into this classification are thrown all of the items whose cause and nature are at all times uncertain. General-expense items consist properly of accessory materials; repair items of small character, such as petty tools; and supply features of any contract. The main items are indicated in Table II.

*The items here given may be considered as contributing materials, inasmuch as they do not form a part of the completed measured quantity unit.

TABLE III

Amount of General-Expense Items on Twenty-One Actual Jobs

Compiled by Method of Percentage of Payroll

Class of Work	Job No	Per Cent of Payroll
Bridge piers—wet	38	17 3
Drag line excavation	71	17 0
Concrete bridge—dry	45	11 2
Concrete bridge—dry	39	9 7
Concrete bridge—small	93	9 7
Concrete wall (wet) and dredging	40	9 4
Wall—wet	39	8 5
Ballast excavation	72	8 3
Series concrete bridges	63	7 7
Concrete bridge	55	6 7
Concrete viaduct	96	6 4
Interurban railway (complete)	48	5 8
Interurban railway (complete)	56	5 6
Pile driving	98	5 2
Sewer	49	5 0
Intake	64	5 0
Concrete arch	62	4 6
Concrete wall	53	4 0
Concrete wall	69	3 7
Dredging	97	2 8
Dredging	92	2 1
Sewer (special)	90	1 7

How General-Expense Items Operate on Particular Jobs. General-expense items really have no salvage value and are a direct charge against each job. Their aggregate cost is of no small consequence, as is shown by Table III, which gives the results of twenty-one jobs. The first item was on a job where the payroll represented approximately 50 per cent of the cost. Therefore this general-expense cost represented 8.65 per cent of the entire cost of material and labor. To show how important these items can be, it only suffices to say that sometimes a contractor would be satisfied with such an amount as a profit.

This table is naturally of much more value to the author than to any reader, because the index number furnishes the key to open up a mental picture of each job and its peculiar features. Such a table of rates cannot be absolute, but requires experience and judgment in its use. It is, however, not different in that respect from the liability company's schedule of rates, Table I, where slight differences in the kind of work or the conditions under which it is

TABLE IV

Petty-Tool Items

Anvils	Pipe cutters
Augers	Pipe dies
Axes	Pipe vise
Bars	Pitcher pumps
Bellows	Planes
Bits	Plastering tools,
Blocks (iron and wood)	Plug and feathers
Boiler tools	Plumb bobs
Boring machines	Punchers
Brads	Push brooms
Brooms	Push carts
Brushes	Pliers
Buckets	Rail
Bush hooks	Rasps
Cables	Ratchets
Canthooks	Reamers
Car movers	Riddles
Car replacers	Rope blocks
Chain blocks	Rope guys
Chain tongs	Salamanders
Chains	Saws
Chalk lines	Scoops
Chisels	Scrapers
Derrick blocks	Screens
Dies	Scythes
Dippers	Sheaves
Dollies	Sheeting caps
Drags	Sheeting pullers
Drills	Shovels
Files	Sledges
Flue cleaners	Spades
Flue expanders	Spike pullers
Flue rollers	Squares
Forges	Stencils
Forks	Stone dogs
Grindstones	Stone hooks
Hammers	Tampers
Hand axes	Tanks
Hand pumps	Templates
Hatchets	Tongs
Hods	Torches
Hose (water; steam, (suction, air)	Track tools (complete outfit)
Jacks	Trench pumps
Lanterns	Trowels
Levels	Trucks
Machinery (special equipment)	Wagon pumps
Mattocks	Well pumps
Mauls (iron and wood)	Wedges
Nippers	Wheelbarrows
Picks	Wire clippers
Pilecaps	Wire rope
Pinchers	Wrenches

TABLE V

Petty-Tool Cost

Compiled by Method of Percentage of Payroll

Class of Work	Number of Jobs	Per Cent of Payroll
Bridges	7	2.6
Viaducts	2	2.0
Interurban railways	2	2.0
Sewers	3	2.0
Walls, etc.	4	2.6
Pile driving and drag-line excavation	6	3.7
Buildings	18	1.8
General average for	40 jobs =	2.14

done make changes in the rates. The percentage given should not be used for estimating purposes.

PETTY TOOLS

Method of Applying Charges for Petty Tools. Nothing with which the contractor must contend is more exasperating than the loss, breakage, stealing, and carelessness in handling of small and petty tools, a list of which is shown in Table IV. This expense cannot be considered as applying strictly to any one job; it is better regarded as a general loss determined by several years' experience.

Table V shows how this cost fluctuates as a percentage of the payroll. It seems advisable, however, to use the average, since jobs overlap; one job may profit by stock on hand, and another may lose on account of new purchases. The term, however, is an absolute general-charge cost and should not be overlooked. The method of percentage of payroll is especially applicable, as the labor employed uses the tools and is directly a measure of the amount necessary.

There is a salvage in Petty Tools which should be taken care of by debiting the Yard account, and this latter account in turn should be credited when the tools are sent out on a new job. It is for this reason that the general average loss is recommended for use as a general charge, although each job should have placed to its account the full amount expended for this item. The salvage is low grade and of but slight cash value.

TABLE VI

Fuel and Oil Items*

Barrel	Engine oil
Battery	Gasoline
Belt dressing	Grease
Boiler compound	Oil faucet
Cans	Oil gate
Coal	Oilers
Coal oil	Packing
Cylinder oil	Power bills
Dry cells	Tallow pots
Dynamo oil	Valve oil
Electrical connections	Waste

FUEL AND OIL

Fuel and Oil Charges Depend on the Job. Fuel and Oil is a direct charge to the work, and whether the amount is large or small depends on the kind of work and the difficulties encountered. Fuel and Oil will be much more of an item for a concrete-bridge pier, requiring pumps, derricks, hoisting engines, pile drivers, mixers, etc., than for a concrete pavement—although the concrete itself may be of approximately the same cost for materials, but not for labor. Therefore, as in other items of general charge, the fact is brought out that the payroll is the best indication of the work, and Fuel and Oil should be apportioned by a percentage thereof.

The items going to make up this division are indicated in Table VI, and the costs by percentages on a number of jobs are given in Table VII.

TABLE VII

Fuel and Oil Cost

Compiled by Method of Percentage of Payroll

Class of Work	Number of Jobs	Per Cent of Payroll
Bridges	7	7 0
Viaducts	2	1 6
Interurban railways	2	5 7
Sewers	3	1 4
Walls, etc.	4	4 5
Pile driving		
Drag-line excavating	6	7 2
Buildings	18	1 4

* Includes any costs of bills pertaining to the production of power Use for distribution of invoices, charging to contract for which purchased.

TABLE VIII

Fittings and Repairs*

Asbestos	Lubricators
Babbitt metal	Machinery fittings
Boiler tools	Machinery repairs
Boots	Nipples
Bushings	Pipe, 3 inches and less
Cable clamps	Pipe fittings
Clips	Plugs
Cocks	Reducers
Couplings	Shaft collars
Crosshead pins	Shafting
Faucets	Shape iron
Flues	Springs
Gages	Steam hose
Gaskets	Siphons
Grates	Threading
Grease cups	Unions
Hose, steam	Valves
Injectors	Water glass
Iron and steel	Wheels
Jet	Etc.

FITTINGS AND REPAIRS

Make-Up of Fittings and Repairs Items. All expense items for material, or labor, or bills for repairs when chargeable to maintenance of plant or equipment should be charged to this account. The idea is to distinguish between expense items directly con-

TABLE IX

Fittings and Repairs Cost

Compiled by Method of Percentage of Payroll

Class of Work	Number of Jobs	Per Cent of Payroll
Bridges	7	1 70
Viadicts	2	.70
Pile driving and drag-line excavation	6	4 40
Interurban railways	2	4 20
Sewers	2	1 30
Walls, etc.	4	1.50
Buildings	18	.67

*Use for distribution of invoices or labor cost, charging to contract upon which equipment is being used. It might seem proper to make this a general charge over all jobs but experience shows that supplies of this class are lost, stolen, or damaged every time a move is made and, therefore, the job should stand the loss and consequent charge.

nected with the particular job and those items which pertain to the general up-keep of machinery. The items ordinarily encountered are shown in Table VIII, and their application to the particular job by the percentage method is shown in Table IX.

COMMISSARY EXPENSE

Items Covered by Commissary Expense. Many times it is necessary to maintain a commissary for the proper care of men. This may be an established camp of shacks or huts, or it may be a train of wagons or cars moving from job to job. The latter sort is suitable for jobs along a railway line. The items to be charged to this account will appear many times in jobs where a job commissary is not existent. For this reason and because of the particular character of the items it would seem desirable to maintain a commissary charge on all work. The items are shown in Table X and the percentage charge to the payroll is given in Table XI.

TABLE X

Commissary Expense Items

Desks	Fuel
Bedding	Ice
Boots	Knives and forks
Buildings	Lighting
Bunk cars	Pots
Boarding cars	Rents
Camps*	Store supplies
Candles	Stoves
Canvas	Table furnishings
Cooking utensils	Tarpaulins
Cots, etc.	Transportation
Covers	Tents, etc , etc.
Food	

TABLE XI

Commissary Cost

Compiled by Method of Percentage of Payroll

Type of Camp	Average Per Cent of Payroll
Permanent Camps	3 5
Moveable Camps	7 0

* Where men must be cared for in camps there is, many times, an extra cost to the work due to installation of camp and losses in operation.

INTEREST

Method of Handling Interest Charges. Assume a contract of \$100,000 to be completed in 10 months, that is, at the average rate of \$10,000 per month. Payments are to be made during the progress of the work to the extent of 85 per cent, payable the 15th of the month following that within which work is done, final payment to be made 60 days after completion. Assume also that profit is at the rate of 10 per cent on cost.

In the first place, at the rate of \$10,000 of work per month the actual average cost is approximately \$9000; the contractor receives \$8500, and \$1500 per month remains in the hands of the client. Then since one-half month's work on the current month elapses before payment is made for the preceding month, \$4500 is expended for cost of work. Since his payroll comes weekly and his average time for bills due is 30 days, the contractor, under these conditions, even if he can buy material on 60 days, is always in arrears and must pay interest on the amount held up in excess of profit and for work done and not yet payable. This charge will amount to a percentage on the total cost, depending on the contractor's credit in the purchase of materials.

The maximum cost in interest is reached where no work is paid for until the whole is completed and accepted. In such a case full interest plus a bonus or fee should be added to the estimate, as no capitalist can afford to speculate on financing work of this kind without receiving at least double the ordinary legal interest for money.

The time necessary to complete work or during which payment is withheld will naturally affect the charge.

MACHINERY AND EQUIPMENT

Division of Construction-Equipment Item. Machinery and equipment can be considered as either an investment or an expense item. In the first case a reasonable interest or rental return should be expected, which may be included in profit. In the second case a rental or depreciation should be charged directly to the job.

Since the Fittings and Repairs account, Tables VIII and IX, is supposed to take care of the wear and tear due directly to the particular job, general depreciation is to be considered under the present heading. If machinery is purchased for any job, one-third

TABLE XII

Construction Equipment*

Air compressors	Excavating machinery
Air tools	Drag-line excavators
Automobiles	Electric shovels
Boilers	Grading loaders
Buckets	Plows
Clamshell	Scrapers
Contractor's	Steam shovels
Orange-peel	Trenching
Cars	Floating equipment
Ballast	Barges
Caboose	Dredges
Concrete	Pile drivers
Dump	Scows
Flat	Steamboats
Freight	Tugs
Hand	Generators
Inspection	Locomotives
Motor—office	Contractor's
Pile driver	Road
Push	Switching
Rail	Motors
Shop	Pile drivers
Supply	Drop
Work	Steam
Concrete hoists	Track machines
Concrete mixers	Ditchers
Derricks;	Layers
Guy	Spikers
Stiff-leg	Spreaders
Drills	Tampers
Engines	Unloaders
Hoisting	Welders
Stationary	Tractors
Elevators	Travelers

to one-half may be charged out at once as depreciation, based upon the cash return from second hand sale.

As the amount of equipment varies with the character of work, the foregoing hints and Table XII are offered as suggestions only in order that this item of general-charge cost may not be overlooked.

MISCELLANEOUS ITEMS AFFECTING UNIT PRICES

Check on Quoted Prices for Material or Equipment. Sometimes general charges, but more often unit prices, are affected by the processes through which the quantity must pass before it is available to the contractor. Consideration should be given to this

* This partial list of equipment is given simply to call attention to this large cost item which may be charged out as depreciation, or rental, or both

TABLE XIII

List of Items Contributing to Unit Cost of Quantity Units F. O. B. Point of Use

Market price
Market price over period of past years
Market price, component parts
Market price, component parts, past year
Base price
Base price over period of past years
Standard extras over base price
Standard extras over base price, past years
Labor supply
Wage scale
Labor troubles
Assembling charges
Packing charges
Loading charges
Lighterage charges
Drayage charges
Freight rates, C.L.
Freight rates, L.C.L.
Express rates
Care *en route*
Car demurrage
Car rental
Unloading charges
Storage charges
Reloading charges
Distribution charges
Unloading charges
Assembling charges
Shrinkage
Commissions
Discounts
Insurance
Interest
Supervision
Overhead expense

feature but in general it serves principally as a check upon the quoted prices for materials or equipment. A list of the elements to be considered is given in Table XIII.

GENERAL OFFICE AND SUPERVISION

This general-charge cost not only varies with the character of work, but also with the volume of work and the magnitude of the organization. The cost exists even if a contractor has "no office but his hat", for he must have some expense. This charge is constant, work or no work, and the amount to be allowed is a matter of individual judgment.

Analysis of Time Sheet. In beginning the consideration of the construction period, reference must be made at once to the Item-

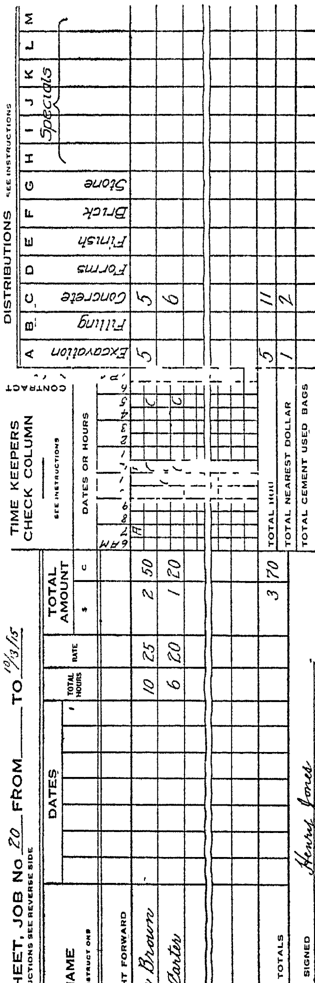


Fig 17 Time Sheet Showing Record for Daily Report For Reverse Side See Fig 19

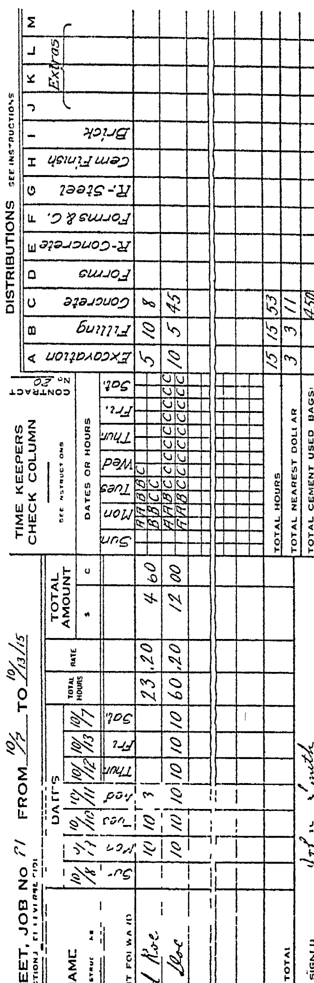


Fig 18 Time Sheet Showing Record for Weekly Report For Reverse Side See Fig 19

ized Cost Estimate and a condensed list made of those main divisions of work which should be carried separately on the regular time-sheet report.

The time sheet prepared is a form which is the development of many years' experience and which is so arranged that it may be used for any sort of work, and carried either as weekly or daily reports to the office or to the contractor, Figs. 17, 18, and 19.

Instructions for the use of the time sheet appear on the right-hand side of the reverse of the sheet, Fig. 19, but, for the sake of clearness they are given as follows:

INSTRUCTIONS FOR USE OF TIME SHEETS

(For Use in Cost-Analysis Timekeeping)

It is intended that this sheet shall be used for all jobs, whether timekeeper makes report to payroll clerk daily, weekly, or monthly, and it is to be used also for making summary reports. In using the time-sheet side, always place date at head of column and when making out summary sheet, place daily dates in the name column. In using the distribution side, the column marked "Timekeeper's Check Column" must have dates placed at the head corresponding to the time-sheet side, and each column represents one day (see Fig 18), the distribution being taken four times per day and marked inside of the four little squares, using distribution letter. However, for daily sheets (see Fig 17), each vertical column represents one hour, and separation of times must be checked to the hour *Distribution columns will be headed by rubber stamp for each job,* and each man's time must be distributed under the proper columns and the totals reduced to dollars and shown at the foot of the sheet, except that for summary sheets, totals will show opposite each day. Timekeepers must keep careful record of cars received, expense, and cement and fill out to form (see Fig 19). Timekeepers and engineers will co-operate to insure that summary sheets are correct as to time distribution and amount of work done Daily time sheets must be turned in to the office at the close of each day, weekly time sheets each Thursday evening, and bi-monthly or monthly sheets will be separated and turned in at least once each week. No change of rate of pay will be allowed between payroll periods.

No deviation from these instructions will be allowed on any job.

It will be noted that the distribution columns are headed by letters. No two jobs may have the same letter for the same kind of work, but for the particular job each letter stands for a classification of work, and the cost-distribution book is similarly headed so that there may be no error.

The time sheet actually used was $8\frac{1}{4}$ inches high by 10 inches wide, and 25 names and 13 distributions were provided for. The sheet can be made $8\frac{1}{2}$ inches by 11 inches, the regular commercial correspondence letter size, and standard letter files used, the time sheets being filed unfolded. When used, the time sheet should be placed in a cover as shown in Fig. 19, which holds the sheet by the

corners in the same manner as an ordinary desk pad, and folds on the vertical center line. The covers, as shown, can be placed in the side pocket of the coat, being $5\frac{1}{2}$ inches by $8\frac{1}{2}$ inches. Fig. 17 shows the front or timekeeping side of the time sheet arranged for daily reports. Fig. 18 shows the same form of time sheet arranged for weekly reports. The top and bottom only of these sheets are shown; the number of lines between can be estimated from Fig. 19.

In Fig. 19 showing the reverse side of the time sheet, special notice is called to the right-hand side where a continuous record is obtained of the job from its beginning to its completion.

Checking the Men. In beginning a job under this system, all the men are given numbers, commencing with the superintendent as No. 1, and identification cards are issued. These cards are retained until the end of the payroll period, generally one week, and surrendered in exchange for the payroll envelope containing wages. Each week new cards of different color should be issued in such rotation that one color will not be used again for at least a month. This scheme avoids the disadvantages of the brass-check system while fulfilling the same purpose. The cards can be expected to remain in good condition for a reasonable period, they serve the purpose of absolute identification, and are receipts beyond question when in the hands of the paymaster.

By using the different colors, there can be no duplication or errors due to the numbers assigned to the men. For instance, No. 72 for the week yellow cards are used may not be the same man as No. 72 the next week when blue cards are issued. This arrangement also makes it possible for the men older in period of employment to be moved forward from time to time, so that the higher numbers will be indicative of new untried men; this groups the steady employes in the tabulation on the payroll. The colors to be used are red, blue, yellow, and green. The men generally wear the cards in their hat bands. The simplification of work to the timekeeper and paymaster is beyond computation.

Payrolls. The time sheets are filled out from day to day and handed to the paymaster, who prepares the payroll. A payroll should be prepared day by day. One hour's work per day will mean seven hours or a whole day by the end of the week, but procrastination may result in failure to complete the payroll on

time, or at least may increase the chance for many careless and unnecessary errors.

The payroll envelopes are then prepared, using the payroll as a basis and these envelopes are made in colors the same as the identification cards; a man who calls for his money several days or weeks after pay day cannot then be confused with any other man of possibly the same number or name working during a different payroll period. During the interval of payroll periods it is necessary to discharge many men, and this proceeding is brought into harmony with the general system by the use of "discharge blanks".

Tracing Employes through Timekeeping System. Let us trace regular and floating employes through their introduction to this system, using both daily and weekly reports.

We will suppose we have two jobs under construction, one on a basis of daily reports and one of weekly. In both cases all men employed have been assigned numbers and cards. The hours, rate, and amount are not filled out on cards unless it is particularly desired to obtain a definite receipt. The time sheets, Figs. 17 and 18, show the men and the time made by them and the character of work upon which they were placed.

Note by Fig. 17 that James Brown, No. 8, worked 10 hours—from 7 a.m. until 12 m., and from 12:30 until 5:30 p.m.—of which 5 were on excavation, distribution letter A, and 5 on concrete, distribution letter C. Peter Carter, No. 9, came on at 11 a.m., and worked one hour until 12 m. on concrete (C), then began again at 12:30 p.m., worked until 5:30, making 6 hours in all. It will be seen that the time spacing at the head of the timekeeper's column means one hour for each division, the upper half denoting the hour and the lower half the half-hour. It is not necessary to mark the distribution letter in each column as it is understood that the man continues on the same work, until by a distribution-letter change at the closest hour or half-hour he is placed on other work.

Assuming at the same time that we have job No. 21 running on a basis of weekly reports, we see Richard Roe, No. 72, Fig. 18, worked Monday to Wednesday, being discharged on the latter day after having made 23 hours as distributed under A, B, and C. His time for Monday and Tuesday having been turned in to the paymaster (possibly the contractor himself), the timekeeper made

out his discharge check for the three hours on Wednesday, and the paymaster added the amount for Monday and Tuesday and paid him $4 60 upon surrender of his card, properly signed. Upon the payroll, the paymaster then writes in the extreme right-hand column "Paid by discharge, No. 72". John Doe, No. 73, on the other hand, having completed a full week's work, is paid, upon surrender of his card, by a payroll envelope of the same name, number, and color, with all proper deductions made.

Ordinarily, it is assumed that large and important work requires daily reports while small work, or jobs where there is little shifting of men, can be reported weekly. In the first case, changes in the distribution may be made to the closest half-hour. In the latter case, changes four times a day would seem sufficient.

This system is flexible and can be applied to any organization, large or small. The cost of installation is not great, and the cost-analysis results more than justify the expense. The rubber stamp used for distribution of time is made so the same stamp can be used on the reverse side of the time sheet, Fig. 19, where the opportunity is given to carry out the total quantities, total payroll, and unit cost for each period, for the previous periods, and for the total to date.

OPERATION OF LABOR-COST DISTRIBUTION SYSTEM

The methods outlined cover briefly the field end of the construction department and its connection with the office, in order to secure cost analysis as to labor distribution. To make the analysis more clear, a view of a reinforced-concrete office building during the construction is given in Fig. 20, and should be studied carefully in order that the proper idea may be had of the many problems presented to the cost-analysis engineer.

Forms and Centers. The view shows seven completed floors of a 12-story office building. The three lower floors have been entirely completed and all the forms and centers removed. The fourth floor is completed but it has been necessary to retain struts under the center of the beams in order to protect them from possible overloading while green. It should be noted that the placing of struts is done after the forms and centers have been removed, and is, therefore, an additional labor charge to Forms and Centers.

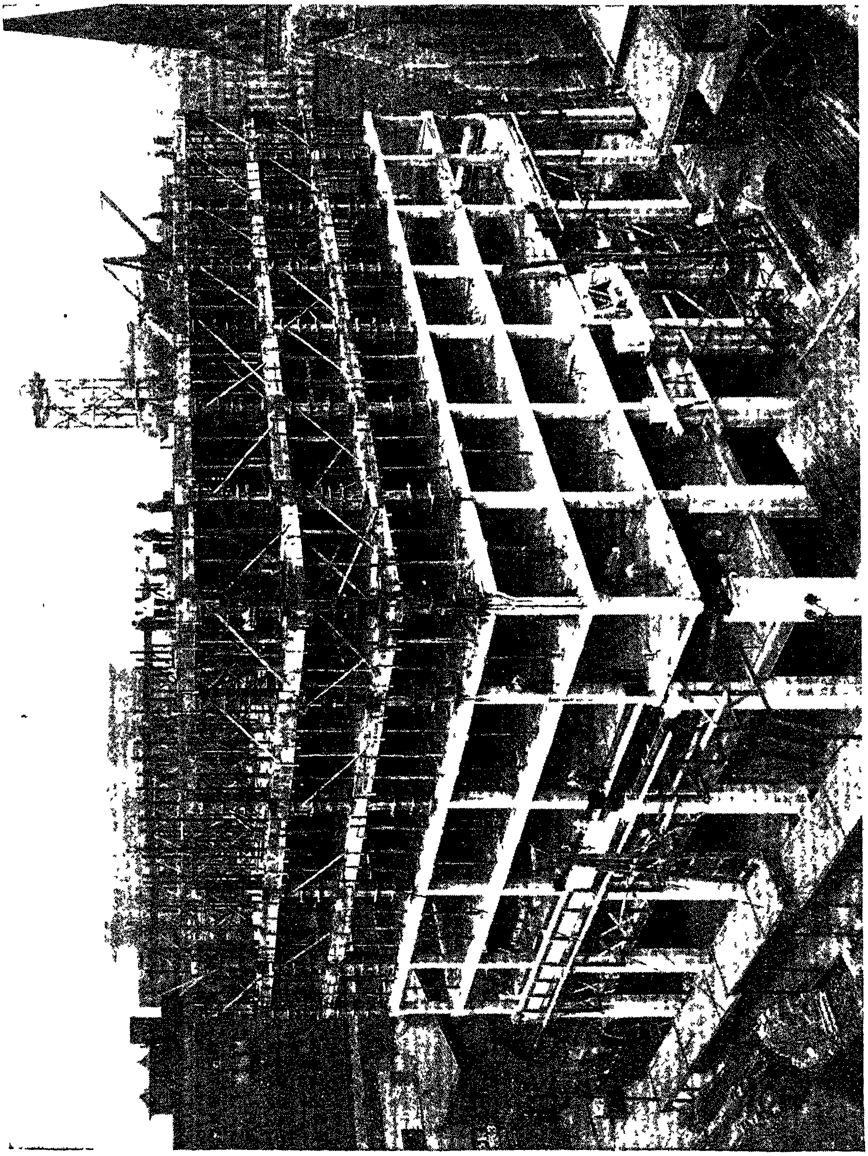


Fig 20 View of Reinforced-Concrete Office Building, Showing Floors in Various Stages of Construction

On the fifth floor the forms and centers still remain, but a part of the supports for centers have been removed in anticipation of the removal of all timber work. On the sixth floor level the timber work is all in place and braced so as to be prepared for the construction of the seventh floor. On the seventh floor forms and centers are completed and concrete work is in progress on the front portion of the building.

In the rear, where the centering work is of extra height, there is to be an assembly hall two stories in height, and for this reason we have a special feature of construction which must be taken into consideration in the final cost analysis.

Checking Labor on Different Classes of Work. The limestone facing of the two lower stories of the building is in progress, and so the timekeeper, in checking up the labor that is employed, must check up the labor of stone masons on the first and second floors; the electric-light workmen and plumbers on the second and third floors; the carpenters removing the forms and centers on the fourth and fifth floors; the carpenters on the seventh and eighth floors; and the concrete workmen in the basement where material is delivered, and on the eighth floor where the concreting is in progress. In connection with this checking of labor he must check invoices and wagon receipts for delivery of reinforcing steel, lumber, tile, cement, and other materials, which may be seen in the material yard in front of the building.

Derrick and Small Hoist. The erection of the small stiff-leg derrick might be considered as a charge against forms and centers inasmuch as it is used principally for the hoisting of timber removed from the floors below. However, since this derrick is used for hoisting reinforcing steel also, the cost of erection and operation of the derrick must be proportioned. In this particular case an electric motor was installed for the operation of the concrete mixer and hoist. On account of mechanical difficulties the motor was abandoned after the fifth-floor level and a contractor's standard double-drum steam hoisting engine was introduced, which may be seen in the right foreground. This engine was used to operate both the concrete hoist and the derrick. Since the concrete hoist was used to raise not only concrete but also tile, brick, and other materials entering into the construction of the building, the cost of these two

installations and their operation must be apportioned against the various classes of work.

Daily Time Sheet. The labor on this building was checked four times a day as to distribution, with the use of the form of time sheet shown in Fig. 18. As the common labor was very largely of foreign character, each man was carried by number—having a numbered card as previously described which was checked at the office in the morning, and brass checks hung on a hook-board with corresponding numbers.

Instructions to Timekeepers. In this case as well as in the case of other complicated structures, the engineer in charge of the cost analysis should issue very definite instructions to the timekeepers and clerks on the job so that proper distributions may be obtained. For example, the ordinary timekeeper cannot be expected to distinguish between that part of the carpenters' work which is designated as Forms and that part designated as Centers, unless the cost-analysis engineer furnishes a detail description of what constitutes each on the particular piece of work under construction. As a further illustration, it will be noted that the concrete on the first two floors, which is very nearly masked by the cut-stone work, has been given a coat of asphalt paint in order that the concrete may not in the course of time stain the stone work. The timekeeper should know whether this painting is a charge against concrete or a charge against stone work. The writer would charge this painting to stone work but others might decide otherwise. If so charged it is an indirect material-and-labor charge against stone work. However charged, the same system should be followed throughout and notations made so that proper comparisons may be drawn with other jobs of like character.

Cost-Analysis Bookkeeping

The independent work of the office contemplates the completion of the records just discussed by the bookkeeping system described as follows:

Ledger Entry System. In the first place, we must have the common double-entry ledger system, such as is used in any ordinary mercantile business. In this ledger are carried the personal accounts of all individuals with whom the concern may be dealing. The

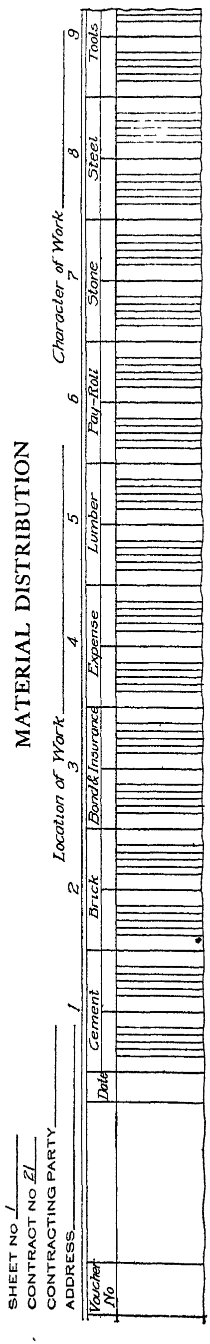


Fig. 21. Typical Material-Distribution Sheet

larger accounts are carried under the firm name, while individual and miscellaneous accounts may be carried under "Miscellaneous Accounts Payable" or "Receivable". In making the ledger entries a vacant line is left opposite the respective debit and credit entries, so that when the account is adjusted an entry will show on both sides of the page. In other words, a vacant line on either side of the page is an indication that the account has not been closed.

Material=Distribution Sheet. Passing this subject of the ledger bookkeeping, which is a more or less purely commercial transaction, let us consider the particular bookkeeping which is applicable to Cost-Analysis Engineering. In Fig. 21 is shown a form for all charges growing out of the operation of the construction organization, the gross amount of which must check with the ledger. This distribution sheet, headed "Material Distribution", serves as an index to all expenditures and all other expenditure records in the office. In other words, while serving as an index or voucher record it gives the distribution of expenditures in such a manner that we may know where and to what account expenditures have been made. In the extreme right-hand column of the sheet—not shown in the figure—the total amounts of expenditures should be recorded, item by item as payments are made, while their distribution may be confined to one column or divided among several. The headings of the distribution columns are written in so as to be adjustable to each contract according to the character of work; particular attention is called to

SHEET NO 1
CONTRACT NO 21
CONTRACTING PARTY
ADDRESS

PAY ROLL DISTRIBUTION

Location of Work — Character of Work

	Pay Roll		6-A Excavation			6-B Filling			Concrete			Forms			6-E Finish			6-F Brick			Stone			6-H			6-K			Cement Used						
Voucher No	Period Beginning	Period Ending	Pay Roll	Quantity	Unit Cost	Pay Roll	Quantity	Unit Cost	Pay Roll	Quantity	Unit Cost	Pay Roll	Quantity	Unit Cost	Pay Roll	Quantity	Unit Cost	Pay Roll	Quantity	Unit Cost	Pay Roll	Quantity	Unit Cost	Pay Roll	Quantity	Unit Cost	Pay Roll	Quantity	Unit Cost	No of Bags	Where Used	No of Bags	Where Used	No of Bags	Where Used	

Fig. 22 Typical Payroll Distribution Sheet Showing Analysis of Column 6, Fig 21

the fact that the columns are arranged for credit and debit entries so that each account or each distribution is complete in its final result.

In case there are more distributions than can be carried on one side of the sheet, the end of the sheet can be cut off and the distributions continued on the reverse side, or on several pages.

At the head of each column appears a numeral indicating the account number for the particular job, the headings of the columns being written in for each job.

Payroll Distribution Sheet. Supplementary to this general distribution sheet appears the "Payroll Distribution" in detail, Fig. 22, which is in the same form as to distribution items as the Time sheet, Figs. 17, 18, and 19.

Particular attention is called to the fact that the gross amount of payroll appears on the Material-Distribution sheet, Fig. 21, which properly may be designated as a general distribution. (On this particular distribution sheet the payroll account appears in Column No. 6.) On the detailed Payroll-Distribution sheet, Fig. 22, appears this same account number with sub-letters heading each column and agreeing with the letters heading the columns on the Time sheet, Figs. 17 and 18, and on the reverse side, Fig. 19. Under this system, while no two jobs may have the same account numbers or letters for the same item, still, if for any reason reference is made to any one particular job, the nature of the account is indicated by the numeral and, in the case of the payroll, this suffix of a letter indicates

a particular classification so that there can be no question or error in the proper bookkeeping or record of costs. The sheets shown in Figs. 21 and 22 are both from loose-leaf books.

Method of Handling Accounts Payable. The method of taking care of the payroll has already been outlined and we therefore proceed to the cost-analysis method of taking care of accounts payable and material bills. It is preferable to have accounts payable handled under monthly vouchers.

ACCOUNTS PAYABLE NO.______

CHK______

A. B. C. CONSTRUCTION COMPANY

TO______________________________DR

____________________ ____________191__

CONTRACT		DISTRIBUTION		DATE	ITEMS	AMOUNT	TOTAL
NO	ACT	ITEMS	AMOUNT				

POSTED ______ B. K AUDITED AND REGISTERED __________ APPROVED FOR PAYMENT __________ SEC'Y __________ PRES

Fig. 23. Typical Voucher Form to Which Is Attached Voucher Check

Voucher Check. Two methods are suggested, of which preference is given to the second. By the first method all invoices from any particular firm for the month are audited and recorded on the face of a voucher form, see Fig. 23, the reverse side of which is in the usual check form. A duplicate of this voucher is prepared with the original by means of a carbon sheet, the reverse side showing distribution. To this duplicate are attached all the original invoices, and thus duplicates and invoices are filed alphabetically, while the original voucher checks are filed consecutively by number. By this method a record is kept both alphabetically and numerically; the two are folded, $3\frac{1}{2} \times 6\frac{1}{2}$ inches, and filed in pigeon holes or boxes.

Voucher Form with Separate Check. The second method, however, is recommended inasmuch as it is sometimes impossible to clean up all accounts monthly. The difference lies in the fact that all invoices are attached to the original vouchers. After they have been submitted and approved, payment is made by check, the voucher having space for voucher number and also for check number, Fig. 24. Since payment is made by check and the voucher remains in the alphabetical file, the same results are obtained as

Audited *C. S.* ACCOUNTS PAYABLE Ledger Page *42*

Approved for payment *D. V. M.* **A. B. C. CONSTRUCTION COMPANY** File No *3*

Month *Oct* 191*5*

To *Smith & Jones* Dr.

Smithville 191

Voucher Envelope. (8½ x 11 In.)

CONTRACT No	CONTRACT ACCT	DISTRIBUTION ITEMS	DISTRIBUTION AMOUNT	DATE OF INVOICE	ITEMS	AMOUNT	TOTAL	CREDITS DATE	CREDITS CK No	RECEIVED PAYMENT
20	4	Nails etc	3.42	10/7	1 Doz Shovels	9 00				
21	9	Shovels "	9.00		1 " B. #12 Wire	2.25	11.25			
21	5	Wire, etc	2.25	10/13	171 #- 8 d.		3.42			
							14.67	10/31	1196	Alonzo Smith

Fig 24 Typical Form Printed on Face of Voucher Envelope in Which Are Placed All Records of the Job

by the previous method, except that we overcome the objection of the banks to handling folded voucher checks and we have the opportunity for the payment of the voucher in installments. The ordinary form of check book is all that is necessary.

Voucher Envelope. A decided improvement over this method consists in having the voucher, Fig. 24, printed on the face of an envelope within which are contained all the invoices for any one particular firm for one month. By this system not only do we obtain a distribution of the account but we have a record of every

invoice. The opportunity is given for a definite statement of the time of payment, check number, and receipt of any one invoice or of all invoices. At the same time reference is made to the page of the ledger upon which the particular account is carried.

If there is no question as to the account, the contents of the envelope need not be disturbed. Should there be any argument, the face of the voucher envelope gives the record, which may be verified by the contents of the envelope; in case there is a question as to payment, an index is given to the different payments and check numbers, which may be investigated to ascertain whether a check was properly received and endorsed. This latter system is recommended as being a combination voucher-and-filing system, adaptable to any office, large or small.

STAMPED ON BACK OF STUB IN CHECK BOOK.

DISTRIBUTION			
JOB NO.	ACCOUNT NO.	TOTAL	
191	6	1000	
	4	.10	
TOTAL		1010	00

DISTRIBUTION			
JOB NO.	ACCOUNT NO.	TOTAL	
191	6	1000	
	4	10	
TOTAL		1010	00

STAMPED ON BACK OF CHECK.

Fig 25 Check Distribution Stamp

Should a creditor ask for an accounting and a checking up of account for any length of time, all that is necessary to do is to refer to the alphabetical file containing his envelopes month by month. An index is at once obtained by which to refer to the numerical file of canceled checks. Under these conditions the ledger becomes simply a permanent book of record, inasmuch as the complete detail appears on the voucher envelope, Fig. 24.

It will be noted that on the Payroll-Distribution sheet, Fig. 22, the column is divided so that in addition to the cost of labor the record of amount of work performed may be entered, so that we may thereby arrive at the unit of cost of the work. From this

record, in connection with the material distribution, we are able to prepare statements at any time showing how the work is proceeding with reference to the original estimate. The necessary information relative to the quantity of work performed is taken from the reverse side of the Time sheet, Fig. 19, supplemented by office records and information not obtainable in the field.

Payment of Accounts. In the payment of accounts the ordinary form of check book may be used, upon the reverse side of the stub and check being stamped a form similar to Fig. 25. At the time a payment is made this form is filled out with the job number and the account number, using the numerical index at the head of the column on the distribution sheets. If necessary, additional information may be shown by indicating under "Items", what the payment is for. Such a check becomes a voucher check which is explained by the voucher envelope on file in the office, and the endorsement of such a check is not only a receipt for payment, but indicates at once the account for which payment was made. Inasmuch as upon the reverse side of the stub appears the corresponding distribution record, the distribution sheets may be posted from the stub book.

This system is not in any sense complicated and may be developed at a trifling expense for printing and rubber stamps; since it is adaptable to any job, large or small, and to any number of jobs, it is deserving of consideration.

The voucher envelope bearing as it does a file number which is the job number, the files for each job may be kept separate. On the other hand, the system may be used alphabetically and all accounts filed accordingly. The latter method is probably preferable, because the information is more readily obtainable and, when the distribution at the left-hand side carries the job numbers, any one account may be carried together for the month, regardless of how many jobs are in progress.

Special Distribution Accounts. *Liability Insurance.* Liability insurance, already discussed fully, requires a detailed payroll distribution, such as Fig. 22, inasmuch as the various classifications of labor are rated differently. The contractor will more than pay for the extra time, trouble, and cost necessary to keep a set of distribution books by securing the benefit of a closer classification of rates.

At any time the liability company's auditor appears, a complete summary of the payroll is shown, with the members of each class of work grouped in the proper columns. By lumping the payroll, it may bear the maximum rate, whereas, if classified, a reduction is secured in the total premiums paid.

Separate Account for Each Section of Work. In case of large construction, the division of the work by sections is of the utmost importance, and it is recommended that each section be considered as a subcontract and the material and payroll costs distributed accordingly. In other words, instead of carrying a job as a complete whole, we divide the total into sections in accordance with the original Outline of Work, and Itemized Estimate, and maintain a separate account for each section so that we have distinct time and work reports and separate distribution sheets.

By adopting this method, the work of the field forces and of the office are harmoniously adjusted and the work of the cost analysis at the completion of the contract is very much simplified.

Cost-Analysis Progress and Final Charts

We will next take up the question of the analysis of the result as shown by the field and office bookkeeping system, with the idea of using progress estimates and progress charts.

Value of Progress Study. Analytical progress study is not only intensely interesting but is of the greatest importance to the up-to-date superintendent, inasmuch as, through the information furnished by the proper recording of the results of the work, he is able to discover the weak points and to direct his best attention to them instead of considering the whole work. Without such detailed information the superintendent is absolutely at a loss to prepare any record; he may be intelligent enough to recognize that certain parts of the work are not proceeding properly, but he cannot be as strong a man as he would be if the record and every part of the work were so tabulated that day by day he could adjust his forces and revise his methods to gain the lowest cost.

Progress Charts. In beginning the construction we have as a basis for our cost keeping and cost analysis the Itemized Quantity Estimate, Fig. 13, and Itemized Cost Estimate, Figs. 14 and 15. In connection with these estimates we have our sketch plans

properly divided into sectional divisions, illustrated by Figs. 8 and 11. We have shown that these sketch layout plans are of benefit in the proper analysis of the work previous to the preparation of the estimate and proposal.

Now, in order that we may conduct our work harmoniously with reference to this preliminary study, these sketch plans must be developed and perfected and general progress charts prepared. These give a picture of the work to be performed, and by the use of colored pencils on the face of the chart the progress of the work can be indicated day by day. In all of the study of Cost-Analysis Engineering, in the preceding discussion, and in the reference to forms for estimates, bookkeeping, etc., the effort has been made to recommend such forms as may be used for different jobs regardless of their character, and there is no doubt but that this can be accomplished.

Features to Be Recommended. When it comes to the subject of progress charts, there can be no standard—as every job must necessarily be handled with reference to the character of the work—but we make the following recommendations:

(1) The general progress chart should consist essentially of a condensed plan, or profile, or both, on a small scale, thereby not only serving the purpose of a progress chart, but also giving a picture of the entire work to be performed, with the various parts in their proper location with reference to the work as a whole. It is needless to say that in making such small plans it is necessary many times to distort certain detail features, but the general layout may be so drawn as to be strictly to scale.

(2) These progress charts should be supplemented, where the work is divided into many sections, by tabular information which would be confusing if the attempt were made to make notations directly upon the plan. This idea is illustrated by Fig. 7.

(3) These progress plans should be supplemented by detail large-scale progress charts for each section of the work. These detail charts need not be finished drawings, in fact they may be very crudely prepared, and generally can be obtained by quick tracings from that portion of the working drawings under consideration. They are to be colored in detail, and should be considered in the cost accounting accompanying the time sheets. Generally speaking, such charts (see Fig. 10) will remain on the work

for inspection during the week and will be sent to the bookkeeper at the end of the payroll period, for a recording of quantities on the payroll distribution book. These detail charts not only serve the purpose indicated of supplying the data for the computing of quantities, but they also serve as a permanent record of the work accomplished during any period; and this information is a great many times of the utmost importance in financial settlement of the contract, especially where the latter goes into litigation.

Analysis of Typical Progress Charts. Sewer Contract. The preparation of the progress chart depends entirely upon the character of the work. Figs. 7 and 8 give an example of the method of handling a sewer proposition. Fig. 10 shows the preparation of the detail progress chart, covering section A, shown in Fig. 8. From time to time the progress of this section is marked by the use of the colored pencils, indicating the advancement of the excavation, concrete forms, back fill, etc. The chart is nothing more nor less than a quick tracing from the working plans and profile with the addition of the 10-foot station lines and the horizontal lines below the base line of the profile for the purpose of providing space to indicate the dates of advancement of each class of work. These sheets also offer the opportunity for notation as to the exact point at which special difficulties or extra work were encountered. From time to time, as these progress reports are turned in at payroll intervals, the general progress chart is so colored as to indicate the advancement of the contract as an entirety.

Reinforced-Concrete Conduit. Fig. 11, in its final form, was supplemented by profiles and tabular information, the same as Fig. 8, but, since in this case the work was of very nearly uniform depth, the width of cut, and character of excavation, different forms of detail sectional progress charts were adopted.

As this conduit was to be built through the principal portion of a large city, it was known that interferences and complications would arise, due to sewers, water and gas pipes, street railways, etc., and that many modifications, changes, and special work would be required. Sheets were therefore prepared, showing the work in plan on a scale of 1 inch to equal 20 feet, Fig. 12, and upon this sheet was indicated the location of these complications as they were encountered.

When the work was completed these detail progress charts with the accumulation of notes made thereon, became a very valuable source of information. They were not only valuable in the final adjustment of the contract, but a copy was furnished the client and served as a permanent record, showing the work as actually constructed in its position with reference to all of the previous underground and surface work.

Miscellaneous Contracts. In the case of a building it is generally possible to combine on one sheet the foundations, floors, and roof by separate sketch plans. These sketch plans may be distorted in scale to show columns clearly, and by a multiplicity of lines around the exterior of the building indicating progress of walls, facing, etc. It is also sometimes advisable to sketch longitudinal transverse sections which constitute in a sense the general progress chart. Naturally, any work such as a building, which is almost entirely above ground, need not be treated as elaborately as underground work where, unless the records are taken during construction, the completed work is covered and concealed for all time.

Bridge work may be treated the same as building work, except that all that is necessary is a plan and a longitudinal section.

Street paving—sidewalk work—yields itself successfully to the method employed for sewer construction, Figs. 8 and 11.

Progress Photographs. At this point it is well to call attention to the fact that no work of any consequence should be constructed without progress photographs. These photographs should be taken at regular intervals, coincident with the end of the payroll period and the coloring of the progress chart. Under this system we thus have a thorough field record of the job: viz, by the time sheet we have a record of the forces employed, the cost thereof, and where such labor was employed; by the colored detail progress chart we have payroll dates, a complete record of the work accomplished, and the progress of the various kinds of work; and by the photographs we have a pictorial record substantiating the first two records.

These photographs should preferably be 8×10 inches and should have marked on the plate the file number, date, name of work, name of contractor, and name of photographer. They should be kept in consecutive order in a binder. In case of

trouble or litigation in making a statement, these photographs may be identified by notations on the back with reference to plans of the work and with reference to time sheets and progress charts.

Progress Estimates. *Material in Hand.* At the end of the payroll period we have in the office, therefore, the time sheets and our progress chart, showing the completed work. The quantities of work to date, as shown by the progress report, are computed, and this may be done very quickly if the Itemized Quantity Estimate, Fig. 13, has been properly prepared in detail. Emphasis is again laid on the fact that this Itemized Quantity Estimate is of constant value as a reference sheet during the progress of the construction, and saves many an hour in supplementary recalculations. The labor cost and these quantities of work are now recorded in the "Payroll-Distribution Book", Fig. 22.

Continuously during the progress of the work the invoices for material purchased have been received; and, where there is any question as to what the material is to be used for, such information has been obtained from the field forces. It is customary to mark invoices as soon as received with a stamp calling for approval, and provided with a space for the information, "For What Used". This detail distribution of material is now posted on the "Material-Distribution Book", Fig. 21.

Final Form of Estimate. From this information we may now proceed to make a progress estimate which should follow the same form as the Itemized Cost Estimate used for the original proposals (see Fig. 14), making changes only in the quantity of work to be done (in case it has been demonstrated that the preliminary estimate was wrong) and in the unit prices in line with the contract prices for material, and the cost of labor as shown by the timekeeper's report. Such a progress estimate serves as an analysis of the work to be done in the light of growing experience on the work itself. The method will give a better idea, than will the regular books, of the actual condition of the work from time to time with reference to its money-making or losing status and the probable outcome.

Value of Progress Estimates. These progress estimates furnish the information as to the probable final outcome, but they also show in detail just where the modifications have occurred, and in what work the gains or losses are being made. This

knowledge is valuable, as it enables us to put our best efforts on the losing parts.

To one familiar with the work, it takes but a short time to prepare such progress estimates; and the one making them will be many times surprised to find that a careful analysis of the work shows his personal observation in the field to have been in error. If the work in the field is proceeding satisfactorily, and excavation is being crowded to keep ahead of other classes of work in which we are possibly more interested, we may lose sight of the cost of this excavation work; but, a study of the progress cost-estimates, may change the methods of excavation or even readjust the progress of the work, in order to keep the cost within reason. In other words, it is possible that a loss on the heavy bulk of excavation, at a few cents per yard, is more than can be overcome by a large decrease in unit cost of the relatively smaller quantities of concrete work which we are pushing at the expense of the excavation cost.

These progress estimates are essential; they are to be prepared in the office, and kept for office records. Construction work is always in a rush; and the activity gives no time for the study of a long column of figures, in order to refresh the mind as to the conditions on any one class of work, or in one section.

Progressive Summary Charts. For this reason, as a last step in the line of progress charts, after the progress sheet has been prepared, a "Progressive Summary Chart" should be made, as shown in Fig. 26. In concise form it furnishes quantities, costs, and dates. It shows on one small sheet how the work is running with reference to the estimate, and what the outcome will probably be. In other words, it is a complete summary of the job, and it may be condensed on a chart so small that one can be carried in the vest pocket. It serves to jog the memory as to conditions. The chart cannot lie, and will not allow the superintendent to deceive himself or to gloss over conditions and make excuses as to costs, either to himself or to his employer. This form of progressive summary along the lines suggested by the writer appeared in its first crude form some time ago in Gillette and Dana's *Cost Keeping and Management Engineering*, and was printed with modifications in *Handbook of Cost Data*. The foregoing outline of progress charts and estimates may seem complicated, but in reality

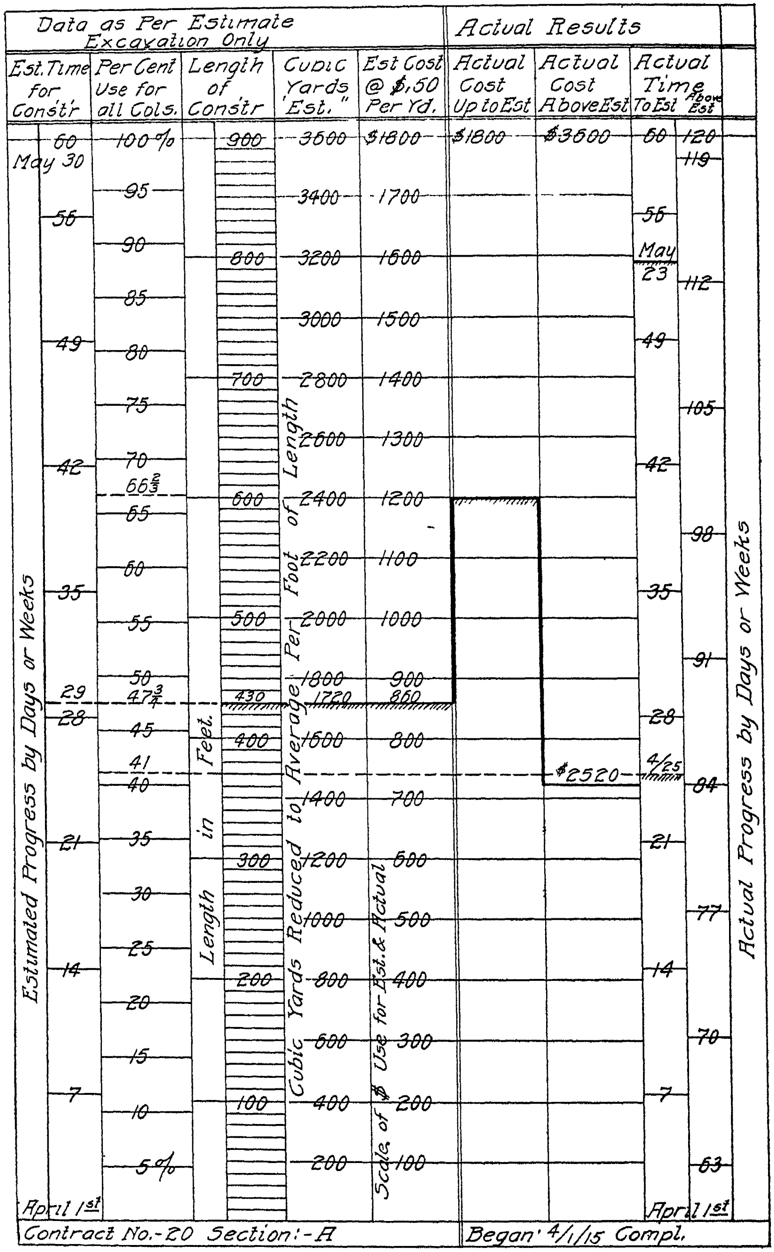


Fig 26 Progressive Summary Chart for Excavation of Section "A", Fig 8

that is not the case. It is not expected that finished drawings be prepared. During the progress of the work calculations must be made continuously; if they are put on scraps of paper, or in a notebook, none of these records are in such shape when the job is completed that they are really valuable for reference; in fact, nine times out of ten they cannot even be found. But by the simple system of progress charts the information is obtained and a record preserved for all time; and it is just as easy to make the notes on a proper sheet as on the back of an old envelope.

Analysis of Progressive Summary Chart on Excavation Job. The Progressive Summary Chart, illustrated in Fig. 26, covers the subject of excavation only, since it is desired to make the explanation of the chart's use as simple as possible. The chart, however, can be extended so as to cover several lines of work on the same or separate sheets. Its use is essential to a complete summary of the work as estimated, and as a record of actual progress. The unique feature, and always the most important one, is the percentage column, by means of which the progress in time, length, quantity, and cost may be compared with the estimate. For instance, in the illustration given, we have in construction a piece of excavation work 900 feet in length, amounting to 3600 cubic yards, which, at an average price of 50 cents per cubic yard, amounts to $1800; and the work can be done in 60 days. All of these items are so adjusted in their divisions by the scale of their corresponding columns that the total appears posted on the 100 per cent line.

The chart is divided into "Data as per Estimate", and "Actual Results". Referring to this, let us consider a certain stage, April 25, when 430 feet in length of the work, begun April 1, have been completed as per the notation made on the chart to this effect. These figures correspond with 1720 cubic yards, which should cost, according to estimate, $860. By referring to the percentage column, we note that this is $47\frac{3}{4}$ per cent of the entire work, and that, according to schedule, we should have been employed upon the work 29 days. The right-hand side, Actual Results, shows us that actual cost has been $1200, or approximately 70 cents per yard; but that only 25 days have been expended. Turning to the percentage column, we note we have expended $66\frac{2}{3}$ per cent of our estimate, 41 per cent of our time limit, and accomplished $47\frac{3}{4}$ per cent of the work.

The right-hand side double columns are for the cost and time and are so arranged that, in case the estimate is exceeded, there will be opportunity for the recording. The illustration just given from the chart, makes it apparent that the cost is exceeding the estimate and will result, if continued, in a final cost of $2520, instead of $1800; at the same time it is also demonstrated that at the rate of progress shown the work will be completed in 53 days, or 7 days less than the schedule. By using the percentage column as a schedule for the comparison of time, quantity, and cost, a complete record is available at all times, together with indications of the probable final result.

The chart may be arranged so as to meet any condition; for instance, instead of length of construction, square feet, or any other unit of measurement may be substituted. The cubic-yard column may be divided in any convenient unit of measurement. Attention should be called to the fact that the column used for cubic yards, Fig. 26, is shown as an average uniform quantity over the entire length of the work. Where the work is very uneven it is perfectly feasible to divide this column, so that the proportions of heavy and light work shall be located exactly as they appear. In other words, this column may be made in a sense a miniature profile of the work, so adjusted that the same results may be attained. This suggestion is deserving of study to the end that the individual student of cost analysis may adapt it to his line of work.

The foregoing description and chart cover only one kind of work, but treat of all elements of that one line. On large and important work such a chart should be made for each classification. On small work several branches can be placed on one sheet, thereby taking care of the entire job.

Final Progressive Summary Chart. The cost of the work is, after all, the main item; and a "Final Progressive Summary Chart", Fig. 27, should always be added to the records, especially for pocket reference. This chart is of slightly different form, in that the quantities are not incorporated; instead, each branch of the work and the total are carried at their estimated cost equal to 100 per cent, with a provision for an increase in actual cost of 50 per cent. The chart shows constantly how each line of work is progressing, and, upon completion of the entire job, shows detail results.

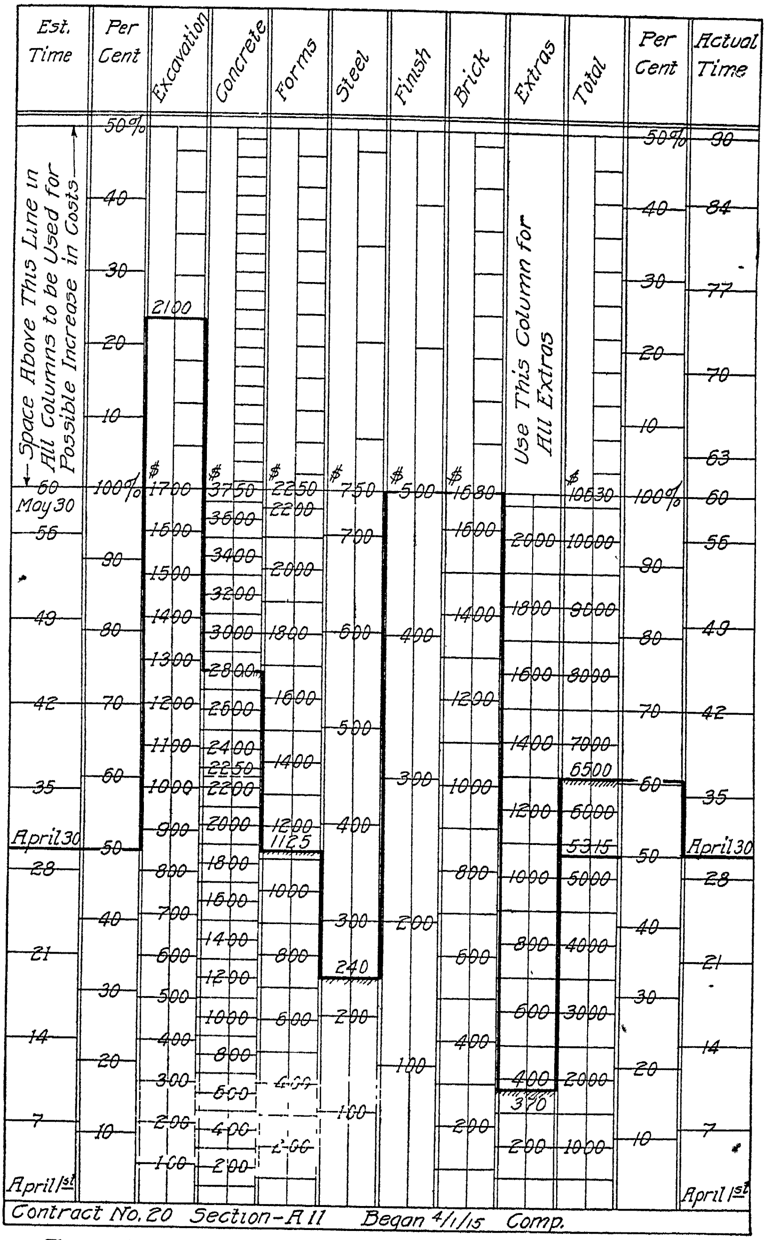


Fig. 27. Final Progressive Summary Chart for Sewer Job Shown in Plan, Fig. 8

Analysis of Chart. Let us suppose the job opened April 1, and that we examine the conditions on April 30. On this date it should be 50 per cent completed; and our cost, to conform to the estimate, should not exceed $5315. We know, however, that our cost is $6500, or 60½ per cent. Why? Referring to the conditions as of April 30, noted on our chart, Fig. 27, we find the excavation complete at a cost of $2100 exceeding our estimate by $400 or approximately 22 per cent. Sixty-two per cent of the concrete has been placed at a cost of $2800, when, according to the estimate, it should have been done for $2250—increase of cost over the estimate for this item of approximately 23 per cent. The cost of forms we find to be running properly with the estimate, at $1125. Fifty per cent of steel labor is estimated at $375. Our cost is only $240—a saving of $135—or approximately 22½ per cent of the cost of constructing 50 per cent of this work. We find that there has developed $370 in extras. Omitting the extras this leaves a net balance of excess cost over estimate of $815.

Readjusting Estimate. Now, assuming at this date that the branches not yet started can be built for the estimate, we make our progress estimate, allowing for increases and decreases, and find as follows:

Items	Cost	
	Increase	Decrease
1. Time O K		
2 Excavation, complete	$400	
3. Concrete, new estimate	917	
4. Forms O. K		
5 Steel, new estimate		$270
6 Finish, assumed O K.		
7 Brick, assumed O K		
Net increase	$ 1047	
Total first estimate	10630	
New estimate	$11677	

The chart enables one to read all this information at a glance, and such detail readings as just given are not necessary in actual use. Differently colored pencils are used for successive dates. The charts can be made on any size sheet, inasmuch as each column is of a different scale in order to reach the 100 per cent line.

It should be noted that Figs. 26 and 27 are constructed on a somewhat different basis. Fig. 26 provides two columns for actual cost, with a possible increase in cost up to twice that estimated. In the make-up of Fig. 27 the 100 per cent line is placed at two-thirds the sheet's height, thereby allowing for a 50 per cent increase of actual over estimated cost. The method is open for selection.

Use of Diagonal Scale in Making Charts. It might be said that the chart is too difficult of preparation, but this is not the case

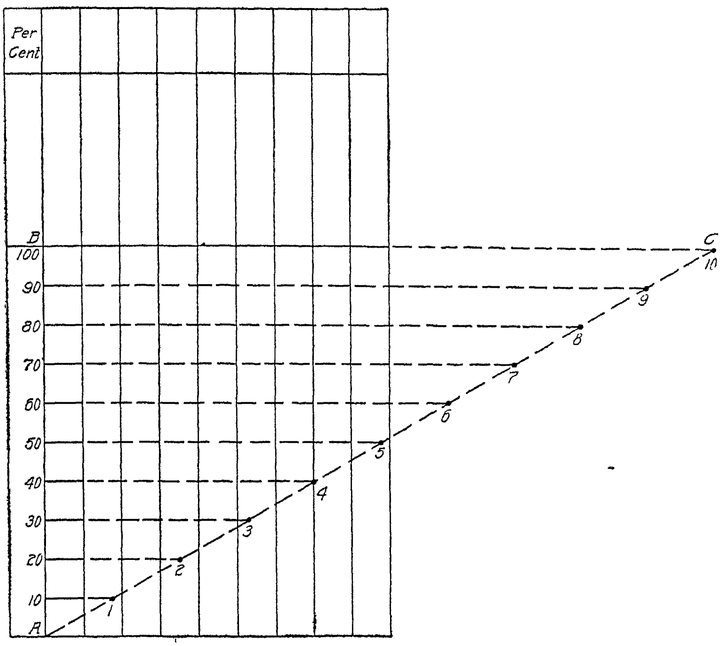


Fig 28. Chart Showing Method of Dividing Distribution Items to Scale

when the diagonal-scale method is used for calculating the divisions of the various columns. In Fig. 28 is shown a chart to illustrate the method. For our purpose we will construct the ten divisions of the percentage column. Extend the 100 per cent line indefinitely to the right, as *BC*. From point *A*, on Line *1* of the column, draw a slant line *AC* by so adjusting 10 equal divisions of the scale placed diagonally that, beginning at point *A*, the tenth division just reaches line *BC* at the point *C*. Mark all ten points and draw horizontal lines to column *AB*. All these lines, except those making the chart, should be in light pencil.

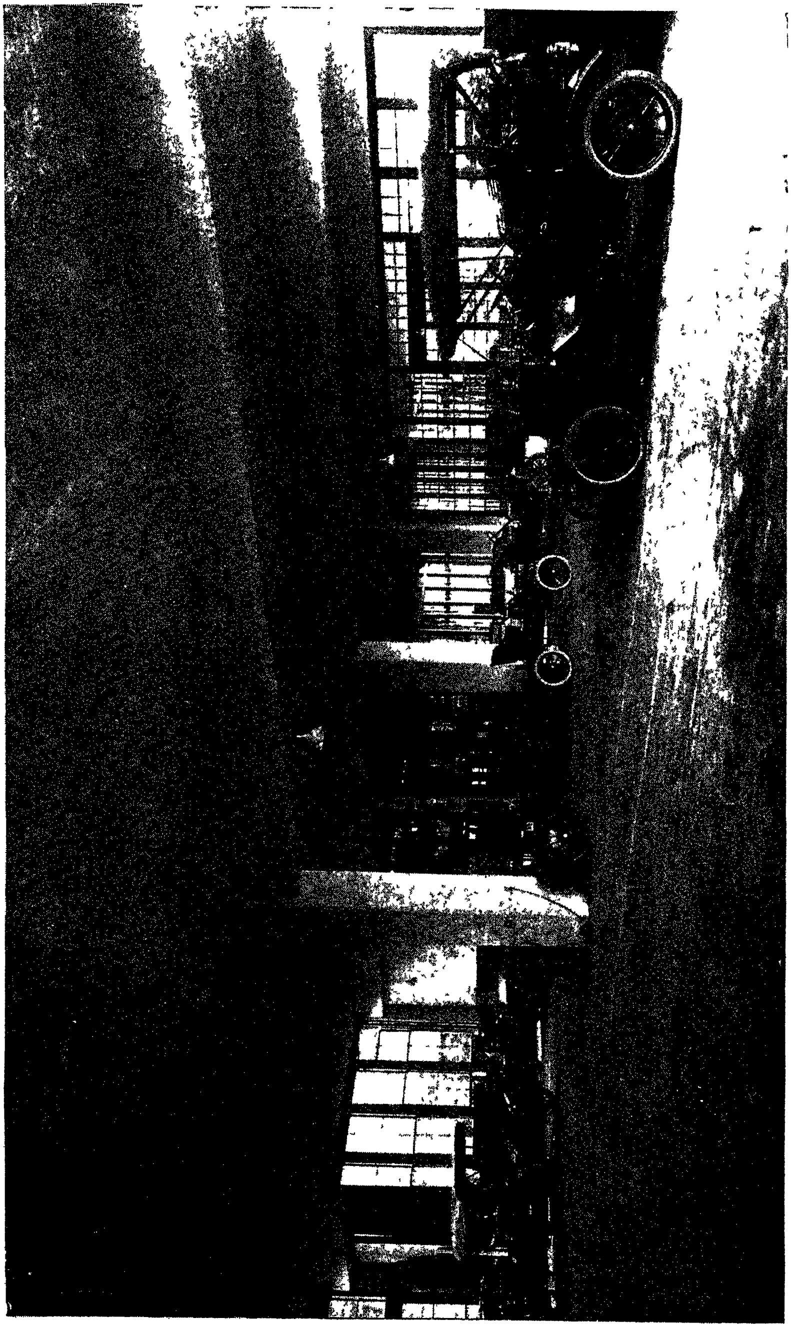


Fig 29. Reinforced-Concrete Building Showing Typical Long-Span Flat-Slab Construction

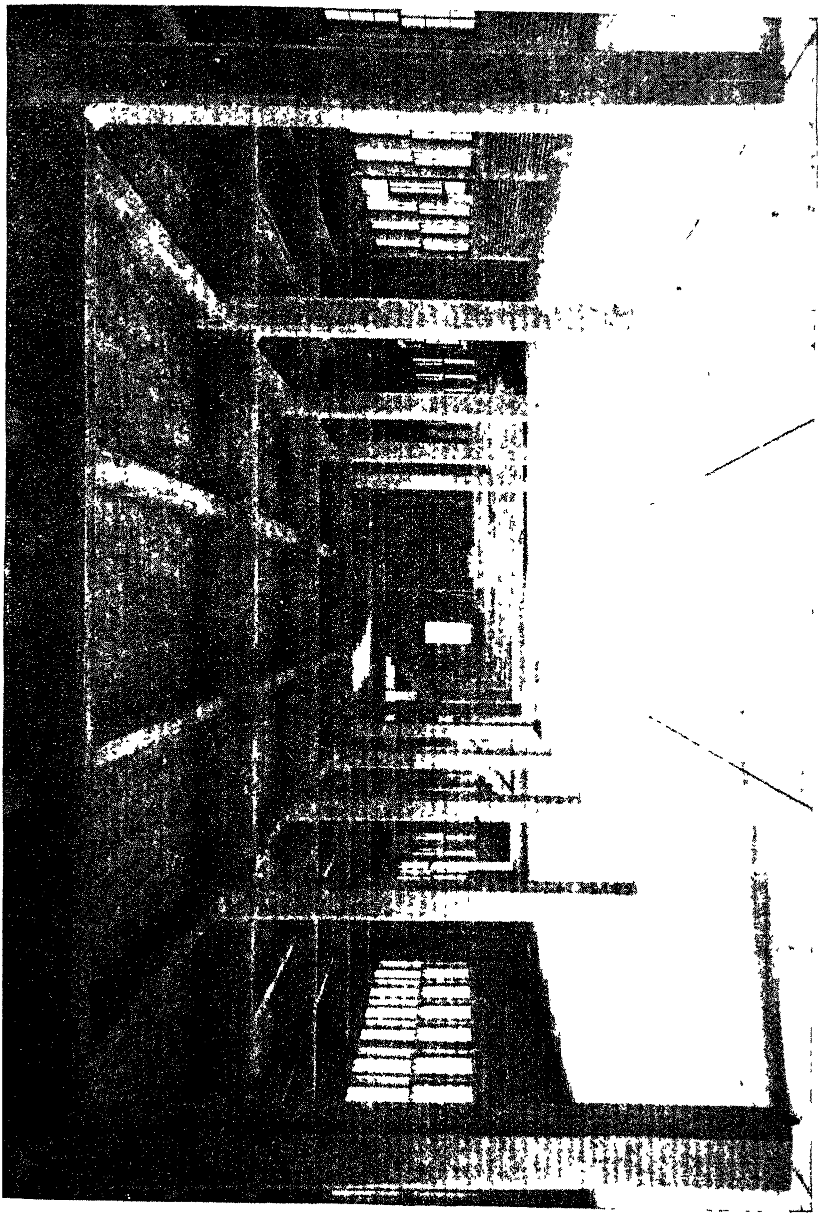


Fig. 20.—Reinforced Concrete Building Showing Short Span Construct

Similar Jobs Requiring Different Cost Distribution. In closing the section on the construction end of our cost-analysis work, it might be well to illustrate how jobs which are more or less similar in appearance, Figs. 29 and 30, radically differ in cost-analysis distribution of labor and material. In Fig. 29 is shown a long-span flat-slab construction, where the amount of forms for the proper construction of the columns and girders is reduced to the minimum, but there is heavy work in connection with the centers to support the long-span slabs forming the floors. In Fig. 30, we have a lighter construction where the forms are largely increased because of the many columns, main girders, and floor beams. In the latter case the timber work necessary to build the forms furnishes to a very large extent the support for the centers.

Both of these buildings were of reinforced-concrete construction, very nearly the same in external appearance, and, so far as the cost bookkeeping was concerned, each would have the same charges for lumber, cement, stone, gravel, sand, tile work, etc. The cost engineer with his system of distribution must analyze these costs and record them in the proper way so that accurate analyses can be provided and future comparisons made.

WHAT IS GAINED BY ANALYSIS?

What Cost Analysis Means. We have reached the point where, in the logical sequence of actual work, our records are completed and we are ready to make a final analysis and summary of our results and experiences. It would seem that few have a proper idea of the meaning of the phrase "Cost Analysis". Time and time again articles on cost analysis have been called to our attention which were nothing more or less than a bare statement of cost records; and although in a great many cases these cost records were divided in an intelligent manner to form the basis of cost analysis, still in no sense could they be said to be real cost analysis. The name implies such a systematic study of results secured on each job, that all forms of digest or cataloging information—as illustrated by differences or inconsistencies—are harmonized, and all jobs brought to a comparable basis. In other words, cost analysis in this final summary eliminates nonessentials, individualities, and specialties; reduces to a uniform basis the pay, hours of

work, etc., to the end that we may reach a point where the results can be tabulated on a comparative scale.

Analysis on Foot-Pound of Work Basis. Such a cost-analysis summary implies also a study of the work accomplished in a broader sense than simply as so many cubic yards of earth excavation, cubic yards of concrete, thousands of brick laid, or feet board measure of lumber. Such an analysis takes into consideration the conditions under which the work was done, giving due credit for location of raw material and methods of operation, extent of work under construction, and the average unit of work.

We are, in reality, approaching the ideal which is to reduce our cost to a basis of foot-pounds of work, although we are doing so only in a very crude manner, since we use ordinary standards of measurement in order to introduce comparisons. It is certainly true that the ordinary laborer is capable of so many foot-pounds of work per day. If his time is expended in doing work at a long distance from the point of supply his energy will be expended in distance at the expense of actual quantities handled. If, on the other hand, the work is arranged either naturally or by proper supervision, so that large quantities can be handled with the minimum of effort, the cost per unit of quantity is correspondingly decreased.

If we should attempt to analyze completely every job within the limits of practical supervision so that we might trace the foot-pounds of work of each operation for each class of labor, we should be doing nothing of real value because of the difficulty and cost of securing information; but it is entirely possible to estimate on a basis of arbitrary units for the different classes of work, and so to secure the same relative comparisons without difficulty. Such arbitrary assumptions, based upon our familiarity with the work in question, furnish a definite basis for recording the information obtained.

ANALYSIS OF ACTUAL WORK

Reinforced-Concrete Building. As an illustration of our idea in this connection, let us consider a job in reinforced concrete. The cubic feet of concrete to be placed in the building, bridge, or other structure, divided by the square-foot area covered, give

the average thickness in feet, of concrete. This is an arbitrary indication of foot-pounds of work necessary, since it indicates, without any direct statement of distance or haul, the foot-pounds of work and energy expended in order to place certain work. We are, of course, assuming that the locations of material supply, equipment, etc., are normal, that is, as closely connected with the actual work as may be usually expected. (Conditions other than this should be specially noted.) The results for reinforced-concrete building work are shown in graphical form in Fig. 31, indicating

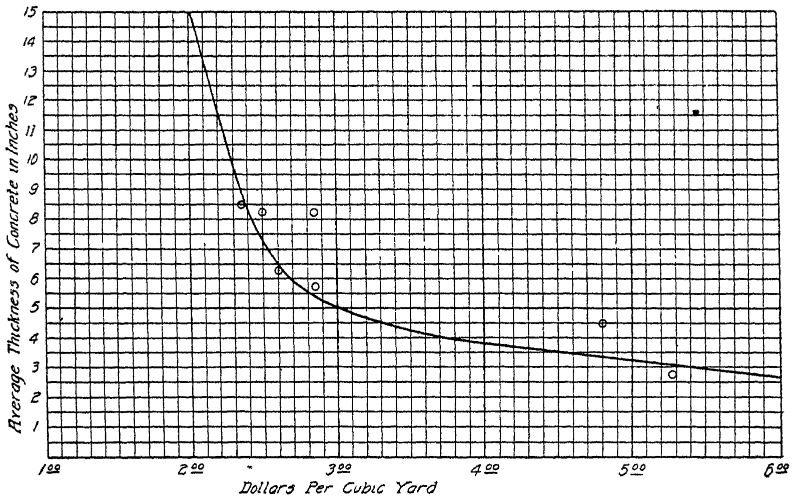


Fig 31 Curve Showing Variation of Cost per Cubic Yard of Reinforced-Concrete Building Work with Thickness of Concrete in Inches

increase in cost of work accomplished, due to distance and to decrease of thickness. The same style of chart may be used for bridge work, pavements, sewers, sidewalks, etc.

Sewer Contract. Fig. 32 gives such a form of analysis carried through the construction of the sewer job shown by Fig. 8, with special reference to the concrete work. The arbitrary unit in this case is the cubic yards of concrete per lineal foot of sewer; in the job there are many different sizes of sewers.

Applying this idea to the construction and placing of steel centers or forms, it is insufficient to say that this cost is so much per lineal foot, inasmuch as we are certain that the cost depends upon the size of the sewer; the next job we have for estimating may give

an entirely different size or a different proportion of sizes. Suppose, however, that we analyze the situation and consider the square feet of forms set, taking the intrados, or, in other words, the inner circumference of the arch, and reduce our cost to a square-foot basis. We find very interesting figures that are immediately valuable for future estimates on any other work. (See Fig. 33.)

Pavement and Sidewalk Work. To a certain extent the same conditions govern in pavement or sidewalk work, both in the foun-

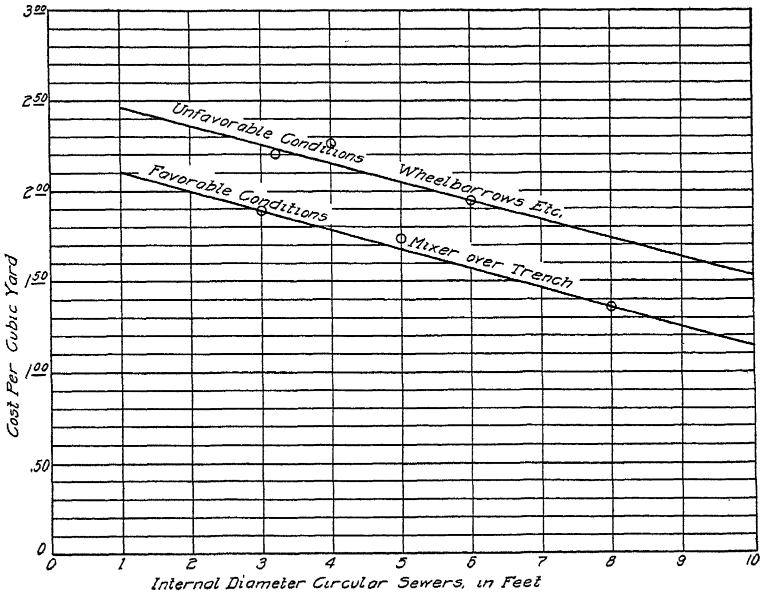


Fig 32 Curves Showing Variation of Cost Per Cubic Yard of Concrete in Sewer Job, Fig 8

dations and finishings. A walk 4 feet wide and a street 25 feet wide cannot be built for the same cost per square foot as a 25-foot walk or a 50-foot street—that is, so far as relates to the unit cost of work accomplished.

General Suggestions. Cost analysis is not cost recording, no matter how elaborated the latter is, or how detailed. Cost analysis is based on cost records properly segregated, but the analysis separates unfamiliar items into a special group by making a note of them, and so divides and classifies the different elements of work

—the quantities, units, hours, and rates—that all jobs may be compared on a certain definite uniform basis.

Each Should Develop Forms for Himself. Much depends on the individual and therefore it is not possible to lay down fixed rules. The illustrations accompanying this paper show how these methods may be applied to practical and actual conditions, but the use of identical forms is not recommended. It has been the writer's desire to suggest rather than dictate, and to stimulate

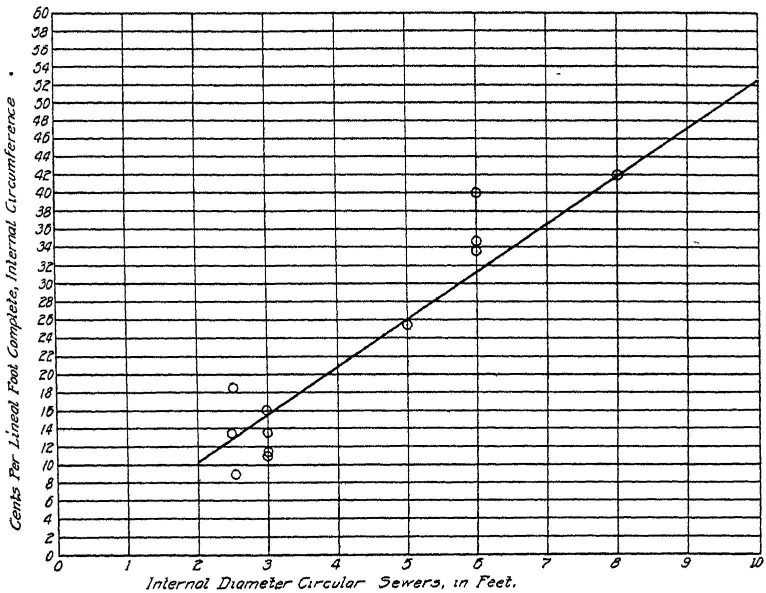


Fig 33 Curve Showing Compilation of Cost Data on Sewer Job for Use on Other Contracts

the cost-analysis turn of mind, which will prompt each man to develop his own forms.

Figs. 31, 32, and 33, illustrative of the final analysis of cost are based upon actual working jobs, and the methods outlined are those developed in actual contracting practice within the writer's own organization. The curves illustrate how the individual may tabulate for future reference the results secured through his own cost-analysis methods.

Charts Up-to-Date. It should be borne in mind that these cost-analysis charts must be kept up-to-date; that is, as fast as new records are secured, they should be plotted upon the charts for

comparison with past work. Either confirming or modifying such experience, they will better qualify the estimator for new work.

Reducing Wages to Uniform Wage Scale. It should also be borne in mind that such a cost-analysis chart must be necessarily reduced to a uniform scale of wages, but this in no way makes it difficult either to prepare or to use in estimating. It is not expected that any job can be carried through at a uniform or definite scale of wages. There will be many rates of pay, and men of different trades, rates, and hours, but in the operation of the payroll the charge of the hours should be carried forward the same as the total dollars, and in this way the average rate of pay is again determined. Every contractor familiar with his own work has a very certain idea about his average rate of pay for labor per hour. However, the payrolls should be totaled for work-hours as well as dollars, thereby giving the average rate for labor.

It is best to prepare the charts on a basis of, say, 20 or 25 cents per hour and then reduce or increase this rate as the case may be. This is, of course, for work of ordinary labor and for very few skilled artisans. To reduce the reading in the chart to a 17½-cent-per-hour basis from the average of the 20-cent-per-hour basis, all that is necessary to do is to deduct ⅛. Fig. 31 shows the results of numerous jobs of reinforced-concrete building work. These jobs covered all designs and classes of structures, viz, light and heavy reinforced-concrete slabs, with and without beams and columns. The average of all concrete over the entire area is indicated by the scale, giving the thickness in inches.

Plotting upon this chart the actual cost per cubic yard of jobs of various character, we arrive at a very satisfactory curve line, indicating what may be reasonably expected in new work. Naturally there are a few abnormal cases, generally caused by special conditions. It should be noted that the average conditions are found between the limits of 5 inches and 8 inches, or an average of 6½ inches at the apex of our curve. Above this limit, as the thickness increases, the cost decreases very slowly. Below this limit, as the thickness decreases, the cost increases very rapidly. This is logical and is an example of the foot-pounds of work accomplished by the laborer.

Discriminating between Classes of Work. In Fig. 32 is another example of sewer construction in which various sizes of the concrete work determine a fluctuating price of labor on concrete per cubic yard. In this case an attempt has also been made to discriminate between (1) that class of work which can be handled over the trench as the work proceeds, the concrete operations being advanced as the work is completed; and (2) work in which the operation must be handled from some little distance and the material wheeled, the plant advancing only at intervals.

On no one of these charts can we expect to arrive accurately at foot-pounds of work. But this is not necessary, for as has been mentioned, we are working along a certain definite line of information, wherein, from the time of the preparation of the estimate until the final cost summary of the job, our methods have been harmonious and on a certain definite basis. To the individual contractor the information compiled is of the most value, as the moment a certain job is called to attention, reference to these charts presents to his mind a picture of the particular job, which is explanatory of the results. For this reason it has been brought out time and time again in this article that the value of the methods suggested lies in their constant adaptation to the individual man's work.

Any System Must Be Consistent. Ruskin says "The crime which bankrupts man and nation is job work, declining from the main design, to serve a turn here and there." Certainly no really good results can be expected from any system of estimating, book-keeping, operating cost keeping, and analysis which is not entirely consistent in all its parts.

Fig. 33 shows the analysis of the cost of installation of steel centers in a number of concrete-sewer jobs. This chart is based upon a table from which we arrive at a nearly uniform price per square foot for centers of any size of sewer within reasonable limits. The square foot is arbitrarily taken as the complete internal circumference of the sewer, the sewers in this illustration being all circular. In order to reduce our figures to as comparable a working basis for estimating as possible, and also to present to the eye as clearly as we can a complete summary, we prepare a chart from which we may take directly in cents per lineal foot the cost for labor on centers for sewers of this character.

The same comments made in the previous case relative to the making of the chart on a uniform rate of pay, apply also to this one.

It should be noted that on all of these charts a definite purpose is attained—a presentation of past experience and future probabilities more graphic and usable than tables of figures. Such a chart, however, should not be used without mental analysis of the new job in question, and results shown must be given careful consideration.

Systematic organization demands recognition of the fact that a system once adopted cannot be modified and must not be disturbed or reorganized because of the individual ideas of any one man. Improvements may be suggested and recorded and tried out as new work requires, but this should be done gradually with correlation throughout the departments. The hardest task for the contractor is to insure that the men employed do not incorporate some particular ideas of their own minds—inventions, or practices learned in previous employment.

The worth of the final analysis in the preparation of these charts depends upon the individual. It would be a very easy matter for the writer of this series of articles to submit thirty instead of three examples, but the principles would not be any further advanced by so doing.

Description of Final Analysis Chart. Fig. 34 illustrates what may be obtained in the final analysis of a construction job by maintaining a proper cost distribution.

It is not generally necessary to prepare such a chart but in the particular case to which this one refers, final adjustment was made through litigation, and in order to present the matter to the court this chart was quickly prepared. It shows the number of men at work each day, the rainfall or snow precipitation, the maximum and minimum temperatures, and the quantity of work accomplished week by week. The work covered by this chart is shown in the photograph, Fig. 20, and particular attention is called to the fact that the statement of conditions is graphically illustrated by the photographs which are numbered and listed on the days upon which taken.

The value of cost engineering is shown in connection with this particular job. When, after completion, it was necessary

to prepare a final statement on account of litigation, it was possible, since the cost distribution had been very carefully kept, to prepare a statement covering the following actual conditions by detail classifications.

FINAL COST-ANALYSIS STATEMENT
Job No. ....

Sheet
1 Men, Hours, and Days worked on Retaining Wall
2 Men, Hours, and Days worked January 18 to April 26.
3 Men, Hours, and Days worked April 26 to September 20.
4 Men, Hours, and Days worked September 20 to December 13.
5 Summary of Quantities and Costs of Materials
6 Summary of Quantities and Costs of Materials.
7 Unit Labor and Material Costs (Averages)
8 Unit Labor and Material Costs (Averages).
9 Quantities and Cost, Deck House.
10 Quantities and Cost, Chimney
11 Quantities and Cost, Meridian St. Sidewalk
12 Quantities and Cost, Retaining Wall
13 Itemized Total Cost of Building and Extras, and Determination of Fixed-Charge Per Cent.
14 Summary of Cost, Fixed Charges, and Totals of Main Building and Extras.
15 Work Accomplished February 1 to April 26, and Unit and Total Costs for Same Period
16 Work Accomplished April 26 to September 20, and Unit and Total Costs for Same Period.
17 Work Accomplished September 20 to December 13, and Unit and Total Costs for Same Period.
18 Summary Sheets 15, 16, and 17.
19 Calculation of Lost Time.
20 Damage and Money Loss.
21 Statement of Cost of Items and Percentages.
22 Final Statement against the Building Company.

Cost-Analysis Phases of Sewer Job. The case instanced above was one which, unfortunately, could not be adjusted without litigation. It should not be understood that cost-analysis engineering tends toward such final adjustments. Many times in construction work there are features of construction which cannot be handled except by actual accounting of material and labor on cost-plus-percentage basis. For example, Figs. 11 and 12 illustrate a job partially on a unit-price basis and partially on a cost-plus-percentage basis, it being necessary to record carefully the cost on each basis.

Unusual Features Encountered. This work covered the construction of reinforced conduits through the down-town streets of a large city. The so-called straight construction work covered something less than one-half of the total cost of construction. The force account, on a cost-plus-percentage price basis, covered slightly over one-fourth of the entire cost. The remaining cost was made up of extras or unusual items which could not, under any conditions, have been itemized had it not been for the efficient system of cost distribution and analysis. The work was a "rush job" and three shifts of labor were employed, working day and night.

The work was divided into sections as shown in Fig. 11, one of these sections being given in detail in Fig. 12. This job would have been a losing proposition to the contractor had it not been for the system of cost keeping and analysis. The straight-contract work covered only those features of the new construction which could be separated and listed on a unit-quantity basis, such as cubic yards of excavation, cubic yards of concrete, and pounds of reinforcing steel. The force-account items were occasioned by unusual features of construction, such as the supporting and protecting of existing water mains, gas mains, and sewers, as they were encountered during the progress of the work.

The contractor would have lost approximately 25 per cent of the total cost of this work had it not been for the system of cost-keeping analysis, whereby, upon the completion of the work, he could present a complete itemized statement of the increased cost of construction which was not included either in the straight-contract items or in the force-account items. These unusual costs covered: increase of labor rates, due to rush work and overtime which necessitated men working at overtime rates and on double shifts; the extra excavation occasioned by caves and reconstruction of water and gas mains and sewers; the increase in cost of excavation on account of increased depth of trenches necessary to pass under obstructions; the change in location of the trench work, causing duplication of work and loss in material and labor; the increase in the amount of lumber and supports, on account of increased depths and changes; the increase in the amount of surplus excavation material, removed because of changes in depth of trenches and

change in style of construction; the increased cost of forms and centers because of change of design; and the demand that the progress of work be doubled, thereby requiring more material to keep in advance of the concrete workers and allow the finished concrete to set.

These items, each somewhat unimportant of itself, in the aggregate amounted to an increased cost, as before stated, of about 25 per cent of the money expended on the job as a whole. A mere statement of this fact would have had little weight with the owners, but there could be no argument when it was presented in detail—with an itemization showing each hour's work and each piece of material, where the material was used, how used, by what laborer placed, and, in addition, not only the day when used but the time during the day when the laborer was actually employed upon this particular piece of construction.

The final report upon this job, including all features, made a book of 184 pages, including blue prints, diagrams, and analysis of construction. The larger number of pages were consumed in itemizing the force account, but, even in such items, it is important that a contractor submit a report to an owner on a contract where the cost-plus-percentage, or cost-plus-fixed sum, is to be the method of payment, so that the owner may know in detail what was done and how; and such a report can be submitted properly only when cost analysis is used to prepare the records secured through cost keeping.

COST ANALYSIS IN VALUATIONS

Importance of Valuation Work. The writer's present position as District Engineer of the Division of Valuation, Interstate Commerce Commission, occasions a reluctance to discuss valuation matters because any discussion of them may be taken as the statement of an official rather than as the personal opinion of an experienced engineer.

However, the subject of valuations, which is now so prominent in the minds of the public as well as with the engineering profession, is of such importance in its relation to Cost-Analysis Engineering that it seems necessary to consider it to some extent. In so doing, it should be *distinctly understood that the statements made*

are not to be connected in any way with the writer's official work with the government.

Scope of Work by Interstate Commerce Commission. For some time past there has been increasing activity in the valuation and control of public utilities by state commissions. By an act of March, 1913, the United States government undertook the physical valuations of the property of all common carriers. The government inquiry to date has covered principally the physical valuation of railroad and telegraph properties. In order to carry out to the fullest extent the work called for by the act of Congress, it may be necessary ultimately to include electric-railway properties, express companies, pipe-line companies, steamship lines, long-distance telephones, and in fact, any and all carriers serving the public in an interstate capacity.

Conditions Governing Valuations. The study of Cost-Analysis Engineering in connection with valuation work presents entirely different conditions than those in connection with the estimating, construction, and analysis of ordinary construction work.

Valuation of New Work or Work in Process. In the case of construction work we proceed to make an estimate based upon the plans and specifications, in which the materials and the labor necessary to install them are described in full detail; from it we may have a fairly definite idea of what is necessary in the way of equipment and costs to assemble the structure as designed. It is true that we must depend upon our imagination to furnish a picture of the finished structure, and of the multitude of details necessary to complete it which are supplementary to the plans and specifications. It is also true that the estimator must fall back on his past experience and again upon his imagination to list and describe the accessory materials and labor required in connection with the construction and work of the various trades. These latter features will not be covered by the plans and specifications, since they are incidental to the work of the several trades, and an accurate estimate can be made only by the man who from experience with the different lines of work knows how much in the way of runways, mortar boxes, scaffolds, temporary lumber, steel, rope, bolts, etc., will be necessary to do that work which finally may be measured as a permanent part of the structure.

To summarize—in any ordinary construction work, we examine the plans and specifications and then form a mental picture of the structure as it will be when completed, but in so doing we have an accurate and full description of all of the materials and the manner of their assembling as they are to enter into the completed structure.

Valuation of Old Work. In the case of the valuation of construction work which is already completed or has been in use for some time, the process is not so simple. For example, we may have a building, which, to the observer, after completion, is a brick-wall structure. The interior is divided into rooms with plastered partitions. Now, as a matter of fact, the brick walls may be, as they appear, a part of the actual load-carrying parts of the building, but on the other hand, the brick may be only a masking or covering material for the steel framework which is the real structure of the building and which in reality carries the brick work as an additional dead load. The partitions in this building may be of tile or they may be of wood studding with wood lath or metal lath; or they may be entirely metal, that is, metal studding and metal lath. In any case, the partitions may be covered with old-style hair plaster or with patent "quick-setting" plaster. The floors may be covered with wood or tile. We may have a reasonable idea of how the floors are constructed, but so far as the observer can see, there is no way of determining just how the floor is secured in position.

Sometimes the plans and specifications for particular structures are available, but in a great many cases they are not. We are therefore dealing with the physical valuation of tangible property for the purpose of considering the cost of reproduction—usually the cost of reproduction of the property as existing at a certain date. Granting that the cost-analysis engineer has had an actual experience in construction work, and that he is dealing with structures where plans and specifications are not available, and where, therefore, the basis upon which the work was done, cannot be known, he is of necessity compelled to draw upon his imagination and experience to furnish a basis upon which the structure might be reconstructed so as to secure practically the same results as he finds, and to reduce to dollars the cost of duplicating the work. This means that, above all, the valuation engineer must use good judgment, and this judgment must be based upon experience.

Depreciation. The subject of depreciation in connection with valuation of property is of extreme importance. It must be given consideration in the making of any valuation, whether the resulting report controls the final valuation or not. Depreciation, when considered in the fullest extent, covers wear and tear, deterioration, inadequacy, obsolescence, and, where the structure is exposed to exceptional hazards, must take casualties into consideration.

A sharp distinction should be made between the wear and tear and deterioration of property, which may prompt the cost-analysis engineer to determine that the structure has a probable remaining life, and the other elements mentioned, which may radically affect the remaining life as so determined. As an illustration: A building may occupy an important business corner in a large city. So far as actual deterioration is concerned the building in question may be good for many years, but obsolescence, when considered in connection with the value of the ground occupied and the business demands of the community, might occasion the removal of the building within a few months.

It is not suggested nor recommended that these elements be given too much importance in field valuation work, but they must be given some consideration and be noted in the final report. The engineer is called upon to report facts to the fullest extent; he should report all facts as he sees them without attempting to make things appear favorable, if in his own mind they are not. There is nothing to be gained by deceiving with optimistic reports, or—to go to the other extreme—with pessimistic reports. The owner or employer is entitled to the truth, and to full information as to how the report is prepared.

What Constitute Valuation Values. Valuation cost engineering, therefore, must pass beyond a mere recognition of the physical elements which enter into a property and which would be reported under cost of reproduction. It must include a consideration of the location, the earning capacity, the investment possibilities, and the going value of the concern under inventory. It probably will not come within the province of the young engineer to decide these larger questions, his work dealing entirely with the physical examination of the construction materials entering into the structure; however, it is very much to be desired that he have the proper

conception of these larger elements of value so that he may correctly note field conditions and enable the more experienced man to place, if necessary, a judgment valuation upon these features.

Forms to Be Used. The forms and blanks used for valuation purposes, especially in the valuation of large properties, must of necessity be so condensed that it is impossible to show in detail all of the elements going to make up the quantity units or the unit prices as adopted. Generally it will be found that the state commissions, in connection with the valuation of public utilities, and the United States government, in connection with the valuation of common carriers, have prescribed standard forms which must be followed in accordance with standard instructions and the standard itemization of accounts. In the progress of valuation surveys, care should be taken to record systematically all the facts and also the quantity units and cost units as finally determined, so that in case it is necessary these calculations may be referred to; this feature should be carefully observed even if these preliminary figures are only in the shape of notations or pencil calculations on scratch-pad paper. The form of the calculation, while it is essential and should be uniform throughout in any valuation work is, however, not so important as to be sure that, by indexing and cross-indexing, the calculations for any portion of the work as shown on the final assembly sheets may be quickly found, in case they are required.

Valuation Problems. Valuation cost engineering has to deal with large and important questions; a few of them are mentioned in order that individual study may be given, although in valuation work the young engineer will in most cases receive specific instructions as to how these matters are to be handled.

Cost of Production. The cost of production may be handled (1) simply as a matter of present materials and present costs; (2) on a basis of the average cost and general methods of construction prevailing over a series of years; or (3) by a thorough consideration of methods, materials, equipment, and costs as they existed at the time of the actual construction of the work. This latter method, in its fullest development, leans toward the appraisal upon a basis of the actual cost to date, which is necessarily an accounting proposition, the data to be secured from the books and

records of the company for verification by inspection of the actual work.

Accounting Problem. This naturally brings to our minds the fact that valuation cost engineering is not alone an engineering proposition but instead is very largely an accounting problem. The ideal combination for this class of work would be an engineer-accountant working in connection with the accountant-engineer. Inasmuch as such combinations do not exist except to a limited extent, the practical basis is the working together of the expert engineer with the expert accountant, so combining their work as to secure a proper report.

Effect of Changes in Neighborhood. Many times, in the appraisal of property, conditions will be found which are not at all the same as when the structures were built. Many times the building-up of the community as a whole has been occasioned by the construction of the property under appraisal. In cases of this kind it must be determined whether the property should be appraised as under the conditions existing at the time of construction or as of the day when the appraisal was made. This line of thought prompts us to consider whether at the time the actual construction was started it was necessary to wreck other and older structures or to move natural formations, which at that time interfered with the construction but which are not now in existence or to be observed.

Effect of Transportation Facilities. If property of some considerable age is under appraisal, consideration must be given present transportation facilities and the transportation facilities at the time of the building's construction. Improvements and availability of materials and supplies must also be studied with reference to both the present time and the time of construction.

Secondhand Values and Quality of Material. The appraiser must take into account the possible use of so-called secondhand material, and also, perhaps, the fact that materials and supplies in the older days were, in general, more carefully prepared and of a higher quality than those now commercially available. As an illustration—consider the much better quality, in dimensions and material, of wood framing-materials entering into structures built, say, 50 years ago.

Overhead Charges. The valuation cost engineer must also consider overhead charges, such as engineering contingencies,

interest, taxes, etc., having also due regard to the time necessary for construction. In doing this, he must pay attention to the conditions existing when the structure was built, and, so far as they may be estimated, those prevailing on the date of inventory.

Appreciation. If it is true that the cost-analysis engineer must give full consideration to *depreciation* and its relations to the various portions of the structure, then he must also give full consideration to *appreciation* in those portions of the property where that may occur. This appreciation may occur because of location or it may result from the peculiar adaptability of the property to the purposes for which it is used. It certainly is true that conditions affecting appreciation must be noted, if depreciation is to be considered.

In the determination of depreciation or appreciation, it is necessary that the individual portions of a property or of any one structure be separated and a weighted average secured. Generally, definite instructions will be given—in the way of a mortality table—as to the life of various classes of material, with reference to their use in the different parts of the structures. These tables can be used as the basis for a judgment report when actual inspection is made to determine whether the particular item should be given a greater or less life than that shown by reported statistics. Many engineers take the position that there is no such thing as depreciation, so long as a property is well maintained. We are not taking issue with this view, but inasmuch as we are here dealing with a question which has not as yet been finally defined or accepted for the purpose of universal practice, it is suggested that the young engineer give careful consideration to the various points mentioned; making his notes accordingly, and having in mind that by so doing the information is made available.

The items mentioned in Tables II to X, inclusive, pages 96 to 102, must be considered in connection with valuation work, and reference should be had thereto. These items are, in general, of such a character that they do not enter into the permanent construction of a structure and cannot be observed, but must be, as before mentioned, estimated on the basis of contributory expense.

Valuation Units. In valuations one of the most important features is the selection of the quantity units and the cost-price

units, and attention is again called to the fact that it is necessary, in order to determine such units, to divide the work under investigation into *cost-analysis sections*.

If the work has been carefully studied and so divided into valuation sections, we may then proceed to select those quantity units and cost units which are applicable to the particular section under investigation. We may recognize two distinct types of units, both for quantity and for cost, namely, simple units and composite or combined units.

Simple Units. A unit for the purpose of a valuation may be considered as that proper quantity unit to which may be applied an ordinary cost unit, at the point of destination or use.

The selection of the quantity unit is more important than the price unit. The first should be a standard, while the latter may be considered a variable, probably determined as the average over a term of years.

Ordinarily, the quantity unit which is adopted will be the natural, customary, or ordinary commercial measure established by United States standard, or by law, custom, trade rules, or local conditions. Sometimes, however, the unit is an arbitrary or selected one, formed by the conversion of a natural unit into another form, possibly for the purpose of pricing all elements on the basis of a common measure.

Illustrations of simple units are lineal feet, pounds, cubic yards of material, rules for brick measurement, or per load or per day for delivery of materials. An illustration of an arbitrary unit might be the case of filling material, foundation concrete, top coat, finishing, and curb construction in sidewalk work, changed to square-foot price; or the material and the carpentry work, and the work of the various trades on a building, reduced to a price per cubic foot enclosed.

In nearly all cases the natural quantity units in valuation work will correspond with those understood in building, market, and trade circles. Sometimes a natural unit becomes a composite unit through custom, such as a cubic yard of masonry. Arbitrary units are generally the result of a selection based upon experience, and in most cases are used for special purposes such as quick estimates or short cuts.

It should be noted that simple or elementary units exist in unit prices as well as in unit quantities. Depreciation on a natural or simple unit may be taken direct.

Composite Units. A composite unit is a combination of a number of natural units to form a certain standard form of construction—or at least a certain type of construction recurring many times—where the composite unit is recognized by the public or by the trade interested as being sufficiently well defined to be self-explanatory and sufficient in detail to serve the purpose of a unit for valuation.

As illustrations there might be mentioned a standard building, a concrete mixer, a car, a plumbing fixture installed, a standard door or window opening complete, a standard sewer catch basin, or a complete unit of any kind recurring frequently in ordinary construction.

It should be noted that composite units exist in unit prices as well as in unit quantities. Depreciation on a composite unit must be a weighted average.

Comparison of Units. Advantages of Simple Unit. (1) The closer analysis of the construction.

(2) The opportunity for a direct, close pricing.
(3) The opportunity for closer comparisons with work of similar character, but with varying quantities or proportions.
(4) The tendency to produce more careful attention to details by field men and computers.
(5) The obtaining of better reference cost data.
(6) The sureness of completeness in final result.

Advantages of Composite Unit.

(1) The opportunity for speeding the work.
(2) The opportunity for criticism on minor points.
(3) Reduction in the volume of work to be done.
(4) Decrease in first cost of labor and dollars.
(5) Establishment of a means for quick, approximate comparisons.
(6) The measuring to the full extent of the experience and judgment of the engineer.

Analysis of Units. In the consideration of units a selection

in many cases will be governed by the elements entering into the construction, and the following features should be considered.

(1) In the selection of unit quantities at point of destination or point of use, analysis should be made of the make-up of the unit, with account taken of the material in its raw condition, the cost of acquisition, labor, and produce, cost of marketing, etc., and of the labor to manufacture, and cost to market the product at each stage of development.

(2) In the determination of the unit cost to install, there should be included the direct labor, indirect or non-productive labor, supervision, injuries, damages, expense items, repairs, rentals or depreciation of equipment, transportation cost, waste, insurance, interest, commissions, discounts, overhead expense, and profit if done by contract.

(3) It should be noted that we may have natural or composite unit prices as well as natural and composite unit quantities. We may have (a) natural simple quantity units at composite unit prices, or (b) composite unit quantities at simple natural prices, or (c) composite unit quantities at composite unit prices.

As illustrations of each of these points, consider the following:*

(a) A barrel of cement, at mill price, plus transportation, plus unloading, plus hauling, plus storage, plus sack loss, plus insurance and interest; (b) standard mill work, at a per annum contract price, f.o.b. location; and (c) a cubic yard of reinforced-concrete culvert, at unit price for concrete, plus unit price for reinforcement, plus unit price for form work.

Other combinations will naturally suggest themselves.

Recommendations. (1) Simple units should be used until the young engineer has proceeded sufficiently, so that composite units may be ascertained with a degree of certainty.

(2) Simple units should be used throughout the early stages of any work, in order to secure working cost-reference data.

*The three unit prices mentioned are variables changing in each case of change of design, regardless of the fact that each unit price is of itself a composite unit.

(3) In general, simple units should be used in all detail reports.

(4) Composite units may be used to facilitate the inventory and to reduce time, labor, and cost in work of extended character, where sufficient duplications occur to make the method worth while, and where specific instructions have been given.

(5) In general, the conditions on any particular job should receive individual study before any decision as to the proper cost units is reached.

It should be added that a quantity unit at a unit price is not necessarily a unit of value. The quantity unit may be readily checked and established. The price unit may include many elements not representing true value or even true cost. A contract price or a quoted cost price should be analyzed with reference to the thing furnished, with due regard to the various processes through which the article has passed before delivery, to determine whether the stated cost, at the cost unit used, is a reasonable cost under the conditions surrounding the transfer of the construction unit. Many times quantity units must be purchased with 80-cent dollars. In some cases this is a natural condition, and represents value received; in many others this is not so.

REVIEW QUESTIONS

REVIEW QUESTIONS

ON THE SUBJECT OF

DAMS AND WEIRS

PART I

1. What do you understand by reverse pressure? Does this pressure favor the stability or not?

2. Discuss the determination of crest width.

3. Of what does the security of dams against sliding depend?

4. What do you understand by theoretical profile?

5. Discuss briefly the two methods of determining stresses in a dam.

6. Make a complete design of a dam 65 feet high on rock foundation. Use both analytical and graphical methods.

7. Discuss Haessler's method of drawing lines of pressure.

8. What might be the effect of a mass of filling against the toe of a dam?

9. Discuss briefly the precautions necessary to take in a dam on pervious material.

10. Discuss briefly the correct design of crests for weirs.

REVIEW QUESTIONS

ON THE SUBJECT OF

DAMS AND WEIRS

PART II

1. Discuss the value of an impervious rear apron.

2. When a dam has a porous fore apron, what is done to assist the stability of the dam?

3. What is the essential difference between arch dams and plain gravity dams?

4. What is an "open" or "barrage" dam and what is the purpose?

5. Discuss the "criterion for safety" of a weir.

6. Discuss the principle of the differential arch.

7. Discuss the principles of design of a submerged weir on sand.

8. What is meant by the term "piping"?

9. Discuss American versus Indian treatment as regards location of draw gates.

10. What are the advantages of open dams as compared with solid weirs?

REVIEW QUESTIONS

ON THE SUBJECT OF

IRRIGATION ENGINEERING

1. Define the term *gravity works* as applied to irrigation, and enumerate the sources of water supply.

2. Explain fully the effect of slope and of area of cross-section upon the location of a canal. What should determine the minimum and maximum velocity of flow?

3. Describe fully the nature of irrigation, and compare the design and control of an irrigation system with the design and control of a domestic-water supply system.

4. Describe fully the principles governing the design and location of masonry weirs and dams.

5. Describe the different types of current meters, and explain their use in determining mean velocity of flow.

6. Describe fully the principles governing the location of an impounding dam, and describe the method of conducting surveys to determine the capacity of the reservoir.

7. Explain the effect of evaporation upon storage-water supplies, and the allowance to be made therefor.

8. Define the term *duty of water* as applied to irrigation. Explain fully how and why the duty of water may determine the financial success or failure of an irrigation project.

9. Define the term *precipitation,* and explain the relation between available precipitation and the necessity for irrigation.

10. Explain the Chezy formula as modified by Kutter.

11. Define the term *lift irrigation,* and enumerate the sources of water supply.

12. Define the terms *wetted perimeter, hydraulic radius,* and *coefficient of friction.*

13. Define the term *weir dam,* and fully explain the principles governing its location and construction.

14. Describe the effect of evaporation upon run-off; and explain how evaporation is affected by topography, soil, humidity, and temperature.

15. Explain the effect of absorption and percolation upon storage supplies, and the allowance to be made therefor.

16. Explain how the variations in run-off will affect the design of the spillway of a reservoir.

17. Explain the method of measuring flow of streams by weirs, and state the Francis formula for the flow of water over weirs.

18. Describe fully the units of measure for water duty and flow.

19. Describe the effect of altitude upon precipitation.

20. Enumerate the parts forming the headworks of a canal, and describe fully the functions and construction of each part.

21. Explain fully the methods of measuring rainfall.

22. Describe the rating of the current meter.

23. Explain fully why the duty of water is rising in some portions of the West.

24. Explain how the topography of the country will affect the location of a canal.

25. Explain the necessity for drainage upon hillside location, and describe the various ways in which it may be accomplished.

26. Define the term *run-off*, and explain how it will be affected by topography, soil, and temperature.

27. Explain the principles governing the flow of water in open channels. Explain the relation between velocity of flow and cross-sectional area of a channel.

28. Describe the field operations in determining the mean velocity of flow in a stream.

29. Define the term *storage works;* explain their functions, and fully explain the principles governing their capacity, location, and construction.

30. What depth of water upon the surface is necessary to water the average soil thoroughly, and how many waterings should be given in a season?

31. Enumerate the parts entering into a perennial canal system, and fully describe the functions of each part.

32. Explain the relation between run-off from a given area and the capacity of storage works.

GENERAL INDEX

GENERAL INDEX

In this Index the *Volume Number* appears in Roman numerals—thus: I, II, III, IV, etc., and the *Page Number* in Arabic numerals—thus 1, 2, 3, 4, etc. For example: Volume IV, Page 327, is written, IV, 327.

The page numbers of this volume will be found at the bottom of the pages; the numbers at the top refer only to the section.

Note —For page numbers see foot of pages

Note.—For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note.—For page numbers see foot of pages.

Note—For page numbers see foot of pages

Note.—For page numbers see foot of pages

F

Note —For page numbers see foot of pages

G

Note —For page numbers see foot of pages.

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages.

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages.

Note —For page numbers see foot of pages.

T

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages

Note —For page numbers see foot of pages.

Note —For page numbers see foot of pages

Note—For page numbers see foot of pages

Note.—For page numbers see foot of pages

Note —For page numbers see foot of pages.